ROOFS

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2

THE

CONSTRUCTION OF ROOFS

OF

WOOD AND IRON

AN ELEMENTARY TREATISE

ON THE

CONSTRUCTION OF ROOFS

OF

WOOD AND IRON

DEDUCED CHIEFLY FROM THE WORKS OF ROBISON, TREDGOLD, AND HUMBER

BY

E. WYNDHAM TARN, M.A., ARCHITECT

AUTHOR OF "THE SCIENCE OF BUILDING," AND "PRACTICAL GEOMETRY FOR THE ARCHITECT, ETC."; EDITOR OF "TREDGOLD'S ELEMENTARY PRINCIPLES OF CARPENTRY," AND "DOBSON'S STUDENT'S GUIDE TO MEASURING AND VALUING"

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PREFACE.

The present little work is intended to introduce the student of architecture to the first principles upon which the roofs of buildings are constructed, and to enable him the more readily to understand the larger and more elaborate treatises upon this important subject. The work is divided into two parts, the first part treating upon the subject of wooden roofs, both ancient and modern, and also upon the method of ascertaining the strains which are exerted upon the several timbers of a trussed roof. The second part is devoted to the consideration of modern roofs constructed entirely of iron, a material now generally employed where wide openings have to be covered; and several examples of iron roofs of different design are described and illustrated, the principles of their construction being also explained.

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PART I.—ROOFS OF WOOD.

PART II.—ROOFS OF IRON.

ON THE

CONSTRUCTION OF ROOFS.

PART I.—ROOFS OF WOOD.

1. THE word ROOF expresses the covering of a house or other building, by which its inhabitants or contents are protected from the injuries of the weather; it also helps to bind together and give firmness to the walls of the structure. A roof is not only an essential part of a house, but it is often made a characteristic feature of its design; as, for example, the roofs of palatial, public, and private buildings in France, which are denominated the "Mansard," and more especially the picturesque roofs of the ancient châteaux of France, and the towers and castles to be found in various parts of Germany; in which much attention was often paid to their ornamentation. The timber roofs of many of the old buildings were made much heavier than was necessary for stability, and we consequently find that they have in some cases thrust the walls out of the perpendicular, where these have not been built of sufficient thickness, or strengthened by buttresses to resist their thrust.

In constructing a roof the object of the builder should be, so to arrange its several parts as to obtain a structure as light as possible combined with a maximum degree of strength.

2. In the present Treatise we shall limit the meaning

of the word Roof to the framing of timber or iron by which the covering itself is supported, and propose to consider the different modes of construction that have been employed in roofs, according to the character and size of the buildings on which they are placed.

In examining the various examples of roofs which have been erected at different times, we cannot help remarking the great variety of *Pitch*, or angle of inclination of the rafters to the horizon, which occurs in them. Thus, in the ancient Greek and Roman architecture, as well as in the modern Italian buildings, we find that the pitch was always what is termed *low*, the angle of inclination being generally less than 30°, or the height less than one-third of the span. This low pitch appears to have been maintained in most European buildings until about the twelfth century, when the taste for roofs of a higher pitch seems to have set in. In England the use of high-pitched roofs was chiefly confined to cathedrals, in which the timber-framing served merely as a protection to the stone vaulting which formed the actual roof of the edifice; the pitch in these buildings was usually one of 60°, the length of the rafter being equal to the span of the roof, and the height three-fourths of the span. In the ordinary parish churches of this country erected during the Gothic period, we seldom find roofs of so high a pitch as 60°, the angle varying between 45° and 55° in the earliest roofs, and seldom exceeding 45° in those of the fourteenth and fifteenth centuries, while many were of a much lower pitch even than this. The covering of these roofs being almost always of lead, a high pitch was quite unnecessary in order to keep out the weather, and was adopted out of purely æsthetic considerations.

In countries where the rains are very heavy, as in ropics, roofs of high pitch are not desirable, as the

water pours off them too rapidly and the gutters are liable to be choked up, thereby causing the interior of the building to be flooded. They are also to be avoided in situations exposed to high winds or hurricanes, as the pressure of the wind on a high roof will often far exceed the normal load laid upon it, and will tend to strip off its covering; also, as the wind presses only on one side of the roof at a time, a racking strain is thereby thrown upon the framing which is extremely dangerous to the security of the structure. In climates where a roof is exposed to heavy falls of snow, a high pitch is generally preferable to a low one, as the snow is thereby prevented from accumulating upon it, but slides off at the eaves; whereas in a roof of low pitch the weight of the snow may become greater than the framing is able to withstand.

The pitch of a roof should also be regulated by the nature of the material used for its covering, as in cases where tiles or slates are employed a very much higher pitch is necessary than where sheets of lead, zinc, or copper are used. The following Table is given by Tredgold to show the maximum pitch to be given to roofs according to the nature of the material of the covering, and also the pressure which such materials produce on every square foot of roofing:—

Kind of Covering.	Inclination to the horizon. Deg.	Min.	Height of roof in parts of span.	Wt. upon a sq. ft. of roofing.
Copper, lead, or zinc .	3	50	$\frac{1}{30}$	copper 1·0 lbs. lead 7·0 zinc 2·0
Slates, large . . .	22	0	$\frac{1}{5}$	11·2
Ditto, ordinary . .	26	33	$\frac{1}{4}$	from 9·0 to 5·0
Stone slate . . .	29	41	$\frac{2}{7}$	23·8
Plain tiles . . .	29	41	$\frac{2}{7}$	17·8
Pantiles	24	0	$\frac{2}{9}$	6·5
Thatch	45	0	$\frac{1}{2}$	6·5
Force of wind, maximum	—		—	40·

The force of wind, which is here put down at 40 lbs. on the square foot of roofing, is in reality about the greatest pressure which in this country it ever exerts on a plain surface at right angles to the direction in which it is blowing; and as this direction is usually horizontal, or nearly so, it follows that the pressure which it exerts on a roof of high pitch must be much greater than on one of low pitch. The reader will find this matter discussed in "The Science of Building," 2nd Edition,* page 188, where it is shown that the pressure on a roof of 60° pitch is three-fourths of that on a vertical plane having the same area, when the wind is blowing horizontally; so that if we take 40 lbs. per foot as the pressure on a vertical surface, that upon a roof of 60° is 30 lbs. per foot; upon one of 45° it is 20 lbs.; while upon a roof of 30° it is only 10 lbs. It will, therefore, be generally sufficient if we include in the pressure of 40 lbs. per foot for the "wind" the effect of "other occasional forces," as the load of snow, for example, which in roofs of low pitch may sometimes amount to a very considerable quantity.

3. We have now to consider in detail the various forms of framing which have been employed in the construction of timber roofs; and these may be divided into two classes, namely, those which consist only of plain sloping beams, called RAFTERS, placed across from wall to wall, on which the covering is laid; and those in which a system of framing called a TRUSS is introduced, upon which the above-named rafters, which receive the covering, are made to rest.

4. The simplest form of sloping roof, and one which is often used over openings of small span, is the LEAN-TO, SHED, or PENT roof, which consists of a number of rafters (Fig. 1) laid across from wall to wall, the ends

* Crosby Lockwood & Co., 1882.

of the rafters being notched down upon square pieces of timber called *wall-plates* laid horizontally in each wall, by means of which the weight of the roof is distributed uniformly along the walls. In this case the

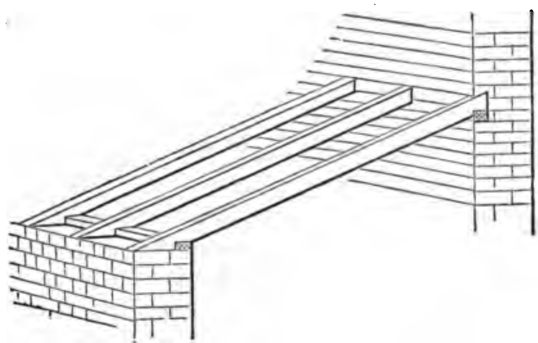


Fig. 1.

rafters are placed about 12 ins. apart, and are laid over longitudinally with boards or battens to carry the slates, tiles, lead, or other covering. Where the roof has a pitch of 30°, the minimum *scantling*, or dimensions of depth and breadth, of each rafter should be as follows: for a *span*, or width of opening between the walls, of 5 ft. the depth should be 3½ ins. and the breath 2 ins.; for a span of 6 ft. the scantling should be 4½ ins. × 2 ins.; for 7 ft. span, 5 ins. × 2½ ins.; for 8 ft. span, 5½ ins. × 2½ ins.

This kind of roof is frequently found over the aisles of old Gothic churches; but when the span exceeds 8 or 10 ft. it is usual to support the rafters upon stronger beams called *principals*, placed at distances apart of 10 or 12 ft. a few inches below the rafters, and having the same slope; the rafters being made to rest upon the principals by means of a horizontal beam called a *purlin*, which is laid across about the midd

of the beams and spiked thereto, the rafters themselves being also spiked to the top of the purlin. Such principals should be at least 7 ins. × 5 ins. for a span of 10 ft.; 8″ × 5″ for one of 12 ft.; 8″ × 6″ for one of 14 ft.; and 9″ × 6″ for a span of 15 ft. The scantling of the purlin should be 7″ × 3″, 8″ × 3″, 9″ × 4″, and 9″ × 5″, respectively, for these several spans.

When we consider the action of the weight of the roof upon the walls which carry it, we see that a certain amount of horizontal thrust is produced upon them, both at top and bottom of the rafters, which tends to push the walls away from each other. If we call W the load on 1 foot linear of the roof, h the vertical height, and s the span, it can easily be shown from the principles of Mechanics that the horizontal thrust of the roof is W multiplied by s and divided by twice h, for every foot length of walling. (See "The Science of Building," 2nd Edition,* p. 117.)

5. The form known as the V or M roof, which is commonly used to cover rows of modern town houses (Fig. 2) is a double SHED roof, in which the feet of the

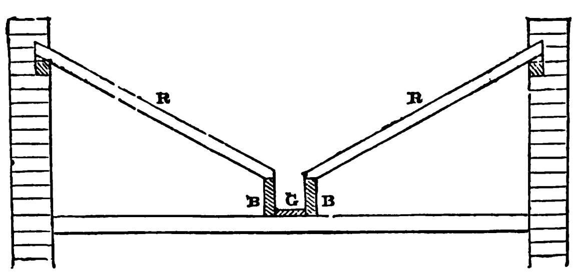


Fig. 2.

rafters (R) rest upon two bearers (B), fixed from back to front of the house, forming with the piece (G) a

* Crosby Lockwood & Co., 1882.

trough-gutter to carry off the rain-water, these timbers resting at their ends upon the back and front walls of the house and upon any cross partition that there may be in the middle. The upper ends of the rafters are notched down upon wall-plates laid in the party-walls.

In such gutters it is advisable to place a grating of wood or metal along the top of the trough, in order to prevent snow from filling it up, so that by resting on the grating it will gradually melt and pass off by the trough below without damaging the interior of the house.

6. When the walls of a building are of equal height and the span between them exceeds 10 or 12 ft., the usual mode of forming the roof is by placing the rafters in pairs (Fig. 3) sloping upwards from the walls to a

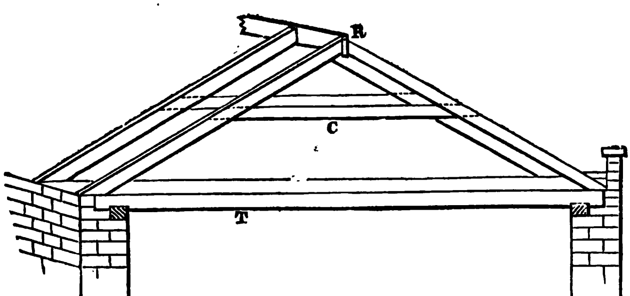


Fig. 3.

ridge, and abutting against a *ridge-piece* (R), which has the effect of keeping the vertices of the rafters all in one straight line, and of preventing the roof from getting out of shape. The feet of the rafters are either notched and spiked to a wall-plate, or else to the ends of the *ceiling-joists* (T), which act as ties to prevent the rafters from pushing out the walls. The thrust of this roof upon the walls can be found in the same manner as was mentioned above (4), namely, W multipli

the half-span and divided by twice the height at the ridge. The scantling of the rafters for a roof of 12 ft. span should be not less than $4\frac{1}{2}'' \times 2''$, and for 16 ft. span $5\frac{1}{2}'' \times 2\frac{1}{2}''$, when the pitch is 30°. In the absence of the ceiling-joists (T), the thrust upon the walls may be partly counteracted by the introduction of the *collar-beam* (C) about half-way up each pair of rafters, being halved and spiked to them in the manner shown (Fig. 4). The scantling of the collar should be the same as that of the rafters.

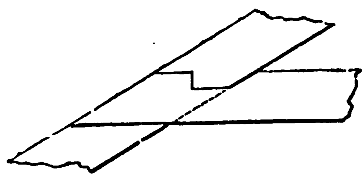


Fig. 4.

7. We frequently find in old Gothic buildings that a modification of the ordinary collar-beam roof is adopted where the span does not exceed 24 ft., as shown by Fig. 5. Here the collar is placed rather higher than the middle of the rafters, and an additional tie is

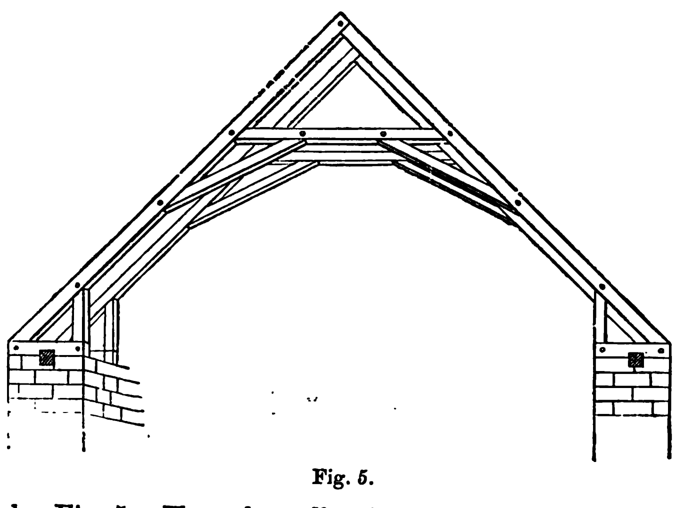


Fig. 5.

introduced in the form of diagonal braces framed into both collar and rafter; the foot of each rafter is also further secured by being framed into a *wall-piece* which lies across the top of the wall, and has a vertical strut framed into its inner end as well as into the under side of the rafter. All these pieces are *mortised* into each other and held together by oak pins driven through the *tenons*. As the timbers are, however, liable to shrink, it is advisable to secure the joints with iron straps or bolts in preference to the pins.

8. When the end of a ridged roof, of the kind we have just described, abuts against a wall, the top of which is built to the same slope as the roof, it is said to be *gable-ended*. But where the end wall is not carried higher than the side walls, the end of the roof is sloped back from the wall at the same inclination as the sides, giving a pyramidal form to the roof, which is then said to be *hipped*, and the external angles formed by the intersection of the two inclined planes are called *hips*, the *hip-rafter* which forms this angle being made stouter than the ordinary rafters, and the short, or *jack* rafters are framed into it. The foot of the hip-rafter is made to rest upon a short piece called a *dragon-tie*, bisecting the angle of the wall and framed into a *diagonal-tie* fixed across the angle and spiked to the wall-plate. In hipped roofs the tops of walls which carry them are at the same level all the way round. When one roof abuts against another an internal angle is formed by the intersection of the two inclined planes, and this is called a *valley*, which is the opposite of a *hip*, and forms a gutter for conveying the rain-water to the eaves.

When a parapet wall, as in Fig. 3, is carried up above the feet of the rafters, a gutter has to be formed between the sloping surface of the roof and the parapet

by means of transverse *bearers* nailed to the feet of the rafters, on which the gutter boards are laid with a proper inclination and with short steps called drips 2 or 3 ins. high every 10 ft. length. The mode of forming these gutters is shown in detail on Plate 28 of the Atlas* to "Carpentry."

9. In roofs of which the span exceeds 20 ft. it is advisable to introduce *principal rafters* at intervals of 10 or 12 ft., which are strong enough to carry the weight of the *common rafters* and their covering. These are placed at the same slope as the roof covering, and support it by means of *purlins*. Thus in Fig. 6

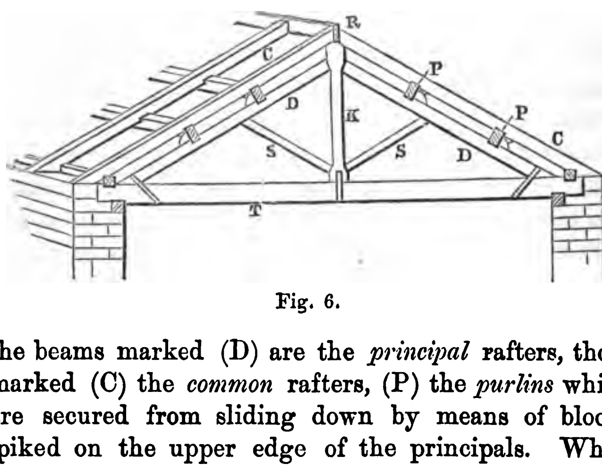


Fig. 6.

the beams marked (D) are the *principal* rafters, those marked (C) the *common* rafters, (P) the *purlins* which are secured from sliding down by means of blocks piked on the upper edge of the principals. When there is only one purlin placed in the middle of the roof, its scantling for a pitch of 30° should be as follows:—With a span of 20 ft., 7 ins. × 3 ins.; span 24 ft., 8″ × 3″; span 28 ft., 9″ × 4″; span 30 ft., 9″ × 5″. The scantling of the principal rafters for these spans should be 7″ × 5″, 8″ × 5″, 8″ × 6″, and 9″ × 6″, respectively. In this kind of roof there is a

* Weale's Series, 182*. Crosby Lockwood & Co.

ridge-piece which rests on the vertex of the principals, and a *pole-plate* on their feet, which support the two ends of the common rafters. In order to prevent the feet of the principals from spreading it is necessary to fix a *collar-beam* about half-way up them, which is halved and spiked to them as before described (6), and shown by Fig. 4.

10. Roofs whose span exceeds 20 ft. require to have the feet of the principal rafters prevented from thrusting out the walls by means of a horizontal piece called a *tie-beam* (T) (Fig. 6), into the ends of which their feet are framed in the manner shown by Fig. 7, being kept in their places either by a bolt of iron passing through them both, or by an iron strap fixed round the end of the rafter, and secured by bolts passing through the sides of the tie-beam. The pole-plate on which the feet of the rafters rest is spiked and notched down on the ends of the tie-beam. The scantling of the tie-beam must depend upon whether it has other duties to perform besides acting as a tie, such as carrying the weight of a floor or ceiling. When it only serves as a *tie*, its breadth should be the same as that of the principal rafters, and its depth just sufficient to prevent it from *sagging*, or bending in the middle. Where only a tie is required, a piece of iron may often be substituted for a beam of wood, being bolted through the feet of the principals; in which case a rod having a section of half an inch is quite sufficient for a roof of 30 ft. span, but in order to prevent it from bending in the middle from its own weight, it is better to make it 2 ins. deep by $\frac{1}{4}$ in. thick than to use round iron.

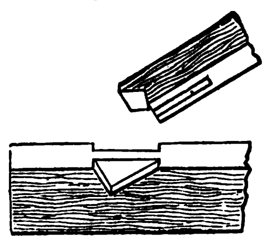


Fig. 7.

11. In order to prevent the sagging of the tie-beam it is usual to frame the heads of the principals into a stout piece of timber (K) which is called the *King-post* (Fig. 6), placed vertically in the centre of the roof with its foot an inch or two above the top of the tie-beam so that it shall not rest thereon. In this position the king-post is firmly upheld by the principals, and if a strap is passed round under the middle of the tie-beam, it can be drawn up until it touches the foot of the king-post by means of key-wedges driven through the strap and king-post. By this means the tie-beam is able to bear four times as great a transverse strain as it did before, and its scantling may therefore be considerably reduced. In order to stiffen the principals, the struts (S) are introduced, which throw part of the strain borne by the rafters down to the foot of the king-post. The whole piece of framing which we have thus gradually put together, and as shown in Fig. 6, is called a *Truss;* and although this is the simplest kind known to the carpenter, it nevertheless embodies all the principles upon which the most complicated pieces of framing are constructed.

The student must be careful to understand that the tie-beam in a truss does not uphold the king-post, but is upheld by it; although it is undoubtedly the case that in most of the tie-beam roofs of the Gothic period the king-post really stood upon the centre of the tie-beam, which was often arched slightly to give it strength. In such roofs, however, the principle of the truss can scarcely be said to exist, the tie-beam being, in fact, what we should now call a *girder*, and made strong enough to support the centre of the roof.

The mode of framing the timbers into the head and foot of the king-post will be found in detail in the Atlas to "Carpentry," Plate XVII. The king-post being ntirely in tension may be, and often is, replaced by a

wrought-iron rod from $\frac{3}{4}$ in. to 1 in. sectional area, in which case a cast-iron head is used to receive the heads of the principals, through which the king-bolt passes, having a head at top and a screw-nut at bottom to hold up the tie-beam through which it also passes.

Rules will be laid down hereafter (18) for finding the scantlings of the several timbers which constitute the framing of a roof, and the following Table gives them for one in which the pitch is 30°, for four different spans; allowance being here made for the strain produced by the pressure of the wind.

SCANTLINGS OF FIR TIMBERS FOR KING-POST ROOFS.

Span.	Tie-beam.	King-post.	Principal rafter.	Struts.	Purlins.	Common rafters.
Ft.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.
20	7 × 3	2½ × 3	4½ × 3	3½ × 3	7 × 3	3½ × 2
24	8 × 3½	3½ × 3	3½ × 4½	3½ × 3½	8 × 3	4½ × 2
28	9 × 4½	4½ × 2½	4½ × 4½	4½ × 3	9 × 4	5 × 2½
30	9 × 5	5 × 3	5 × 5	5 × 3	9 × 5	5½ × 2½

In this table it is supposed that the truss carries only one purlin on each side, which is placed at the middle of each principal; but where two purlins are used on each side their scantling, as well as that of the common rafters, may be reduced. If the tie-beam has no ceiling to carry, its depth may be reduced one-half.

The following are the strains borne by the several members of this truss: the principal rafters (D) are subjected to a transverse strain, and also to a longitudinal compression; the struts (S) are entirely in compression in the direction of their length; the king-post (K) is entirely in tension; the tie-beam is in tension, and is also strained transversely by its own weight and that of any load it may have to carry.

12. When the span to be covered by a roof exce-
30 ft., it is advisable to hold up the tie-beam in

than one place, which is done by means of two upright pieces called *Queen-posts* (Q) (Fig. 8), these being them-

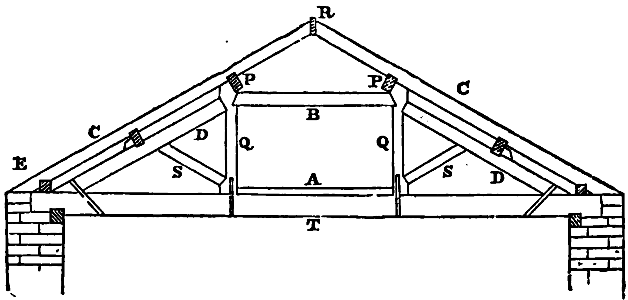


Fig. 8.

selves upheld by the principals (D), and by the *straining-piece* (B), which are framed into the heads of the queens. The horizontal portion of the thrust from the struts (S) is counteracted by the *straining sill* (A) placed between the feet of the queens. The strains on the several pieces are the same as in the king-post roof (11), the additional pieces, A and B, being entirely in compression. In a roof of this kind at least two purlins are required on each side of the roof, which generally divide the length of the rafter into three (or more) equal parts. The following Table gives the scantlings of the several pieces which form a queen-truss, the pitch being taken at 30°, and two purlins being provided on each side.

SCANTLINGS OF FIR TIMBERS FOR QUEEN-POST ROOFS.

Span.	Tie-beam.	Queen-post.	Principal rafter.	Straining beam.	Struts.	Purlins.	Common rafters.
Ft.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.
32	7 × 4	4 × 3	4½ × 4	6 × 4	4 × 3	7 × 3	3½ × 2
36	8 × 4½	4½ × 3	5 × 4½	7 × 4½	4½ × 3	8 × 3	4 × 2
40	9 × 5	5 × 3½	5½ × 5	8 × 5	5 × 4	9 × 4	4½ × 2
45	9 × 5½	5½ × 4	6 × 5½	9 × 5½	5½ × 4½	9 × 5	5 × 2¼
50	11 × 6	6 × 5	7 × 6	10 × 6	6 × 5	9 × 6	5½ × 2½

Robison also recommends the form shown by Fig. 9 as a proper construction of a trussed roof, preferable to that which is generally used, and the king-post which is placed in it may be employed to support the upper part of the rafters, and also for preventing the straining beam from bending in either direction in consequence of its great compression. It will also give a suspension for the great burdens which are sometimes necessary in a theatre. The machinery has no other firm points to which it can be attached; and the portion of the single rafters which carry this king-post are but short, and therefore may be considerably loaded with safety.

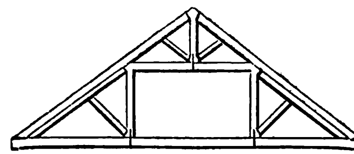


Fig. 9.

13. The nature of the material used as a covering to a roof must always in some degree affect its mode of construction. When it is intended to be covered with lead or zinc the rafters must be boarded over with deals laid close together, either horizontally or diagonally, on which the sheets of metal are secured by means of *rolls* of wood 2 or 3 inches in diameter spiked to the boarding every 2 or 3 feet length of roofing, the rolls being laid straight down from the ridge to the eaves, and the edges of the metal lapped over them so as to keep out the weather. In some cases the common rafters are entirely dispensed with, and the boards laid down the roof and nailed to the purlins; when this is done the purlins must be placed nearer together, or else the boards must be made much stouter than in the case where rafters are provided.

In order to fix slates or tiles on the rafters it is necessary to lay narrow battens longitudinally at

tances apart, regulated by the gauge of the slates or tiles; the feet of the slates being tilted up by means of a wedge-shaped or *feather-edged* slip of wood laid along the lower edge or *eaves* of the roof. Wherever the slating abuts against a wall it is necessary to lay a similar slip of wood called a *tilting-fillet*, to throw off the rain-water. If the ridge or hips are covered with lead or zinc, a rounded roll of wood is spiked thereto, over which the metal is dressed, and to which it is spiked.

Where close boarding is laid upon the rafters it is a good plan to cover it with *prepared felt* before putting on the outer covering, as this material prevents a good deal of the sun's heat from penetrating into the rooms immediately under the roof, and also keeps them warmer in winter. The felt will also assist in keeping out snow or rain which may be driven under the laps of the slates or metallic covering.

14. Before proceeding to the consideration of other forms of roof-framing, we will give the late Professor Robison's method of ascertaining the strains and thrusts produced in a king or queen truss, without resorting to the aid of mathematical formulæ.

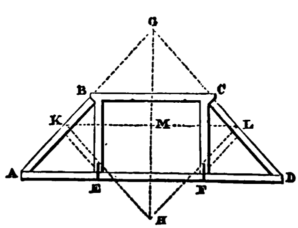


Fig. 10.

Let it be required to find the horizontal thrust acting on the tie-beam AD of Fig. 10. This will be the same as if the weight of the whole roof were laid at G on the two rafters GA and GD. Draw the vertical line GH. Then, having calculated the weight of the whole roof that is supported by this single frame ABCD, including the weight of the pieces AB, BC, CD, BE, CF, themselves, take the number of pounds, tons,

&c., which expresses it from any scale of equal parts, and set it from G to H. Draw HK, HL, parallel to GD, GA, and draw the line KL, which will be horizontal when the two sides of the roof have the same slope. Then ML measured on the same scale will give the horizontal thrust, by which the strength of the tie-beam is to be regulated. GL will give the thrust which tends to crush the rafters, and LM will also give the force which tends to crush the collar-beam BC.

In like manner, to find the strain on the king-post BD of Fig. 11, consider that each brace is pressed by half the weight of the roofing laid on BA or BC, and this pressure, or at least its hurtful effect, is diminished in the proportion of BA to DA, because the action of gravity is vertical, and the effect which we want to counteract by the braces is in a direction E *e* perpendicular to BA or BC. But as this is to be resisted by the brace *f* E acting in the direction of *f* E, we must draw *f e* perpendicular to E *e*, and suppose the strain augmented in the proportion of E *e* to E *f*.

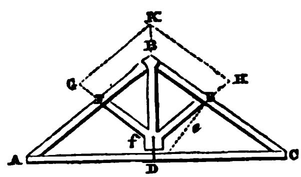


Fig. 11.

Having thus obtained in tons, pounds, or other measures, the strains which must be balanced at *f* by the cohesion of the king-post, take this measure from the scale of equal parts, and set it off in the directions of the braces to G and H, and complete the parallelogram G *f* HK; and *f* K measured on the same scale will be the strain on the king-post.

The builder may then examine the strength of his truss upon this principle, that every square inch of oak will bear at an average 7,000 lbs. compressing

stretching it, and may be safely loaded with 3,500 for any length of time; and that a square inch of fir will in like manner securely bear 2,500. And, because straps are used to resist some of these strains, a square inch of well-wrought tough iron may be safely strained by 20,000 pounds.

15. The method which is now generally employed for ascertaining the proportionate strains upon the various parts of a truss, is by forming a diagram of lines drawn parallel respectively to the directions in which the several forces act. The following example will show the application of this method to a king-post roof, and also how the strain produced by the pressure of the wind acting only on one side of the roof at a time can be estimated.

Let Fig. 12 represent the truss having a span of 20 feet, the rafters being inclined to the tie-beam at 30°, and the trusses supposed 10 feet from centre to centre; the purlins will throw the weight

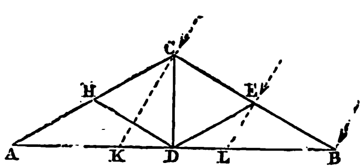


Fig. 12.

on the points A, H, C, E, and B. Assume that 20 lbs. per square foot, measured on the slope, is the weight of the roof timbers (except tie-beam and ceiling) together with the covering and snow; then 4,600 lbs. is the weight borne by each truss; the weight of tie-beam and ceiling 2,400 lbs. Let the force of the wind be 40 lbs. per foot acting perpendicularly to the slope of the rafters on one side only; this will amount to 4,600 lbs. uniformly distributed over one side of the roof and at right angles thereto. Now, when a continuous beam is uniformly loaded and supported at the centre and two ends, it is generally reckoned

that $\frac{3}{16}$ths of the load is borne at each end and $\frac{5}{8}$ths at the middle. Hence the loads at A and B are each $\frac{3}{32} \times 4{,}600 = 430$ lbs.; at C, $\frac{6}{32} \times 4{,}600 = 860$ lbs.; at E and H, $\frac{5}{16} \times 4{,}600 = 1{,}440$ lbs. Also the weight of tie-beam and ceiling produces a load of $\frac{3}{16} \times 2{,}400 = 450$ lbs. at A and B, and of $\frac{5}{8} \times 2{,}400 = 1{,}500$ lbs. at D. The force of the wind on the right-hand side produces at B and C a pressure of $\frac{3}{16} \times 4{,}600 = 860$ lbs., and at E $\frac{5}{8} \times 4{,}600 = 2{,}880$ lbs.

We have now to draw two stress diagrams, one showing the stresses arising from the vertical forces, and the other those produced by the pressure of the wind acting at right angles to one side of the roof.

Half the total weight of the roof is of course borne at each end A and B, and amounts to 3,500 lbs., which is the reaction at A and B.

To construct the diagram, Fig. 13, for vertical forces draw a vertical line *c b*, and measure *a b* representing on any scale 430 lbs., the pressure at A or B; take *b c* equal to 3,500 on the same scale; draw *c d* parallel to AB, and *a d* parallel to AC. Measure *a e* equal to 1,440 lbs., the vertical pressure at E or H, draw *e f* parallel to AC, and *d f* parall-

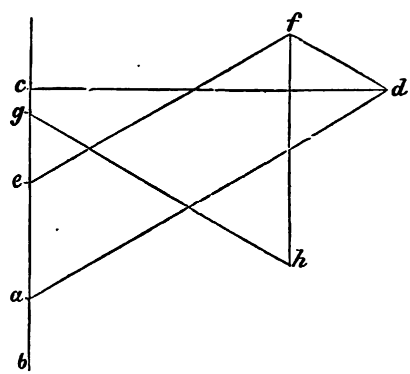


Fig. 13.

to DH. Take $e\,g$ equal to 860 lbs., the load at C, and draw $g\,h$ parallel to DH. Then $c\,d$, $d\,a$ represent in direction and magnitude the stresses in AD, AH respectively, the former in tension and the latter in compression; $d\,f$ and $f\,e$, those in HD, HC, both in compression; $f\,h$, the tension in the king-post. Measuring these lines by the scale, we find $c\,d = 4{,}500$ lbs. the tension in AB; $d\,a = 5{,}250$ lbs. the compression in AH or BE; $d\,f = 1{,}400$ lbs. the compression in HD or ED; $f\,e = 3{,}800$ lbs. the compression in HC or EC; $f\,h = 2{,}900$ lbs., the tension in the king-post CD.

We have now to find the stresses arising from the wind acting in the direction of the arrows at B, E, and C, at right angles to BC. The pressure at C is 860 lbs., and produces a reaction at B equal to $860 \times \frac{AK}{AB}$, or 290 lbs.; also a reaction at A of $860 \times \frac{BK}{AB}$, or 570 lbs. The pressure at E is 2,880 lbs., and produces a reaction at B of 2,880 lbs. $\times \frac{AL}{AB}$, or 1,900 lbs.; and a reaction at A of $2{,}880 \times \frac{BL}{AB}$, or 970 lbs. Therefore the total reaction at B is $860 + 290 + 1{,}900$, or 3,050 lbs.; and at A it is $570 + 970$, or 1,540 lbs. Draw a line $q\,l$ (Fig. 14) parallel to CK, and take $k\,l$ to represent 860 lbs., $l\,m$ to represent, on the same scale, 3,050 lbs.; draw $m\,n$ parallel to AB, and $k\,n$ to BC; then $m\,n$ repre-

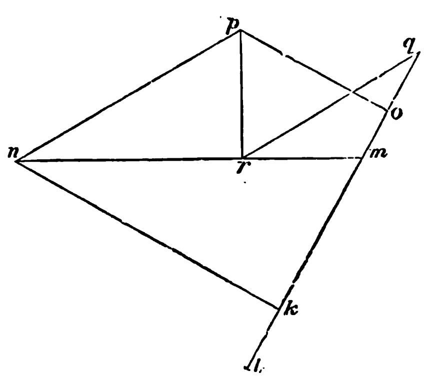


Fig. 14.

sents the tensile strain in AB, $n\,k$ the compressive strain in BE. Take $k\,o$ equal to 2,880 lbs., and draw $o\,p$ parallel to EC, $p\,n$ parallel to ED; then $o\,p$ represents the compression produced by the force of the wind in EC, and $p\,n$ that in ED. Draw the vertical $p\,r$, which is the strain in the king-post; and draw $r\,q$ parallel to AC, which is the compression produced in that rafter. Measuring by scale, we find $m\,n$ is 4,400 lbs., the tension in the tie-beam; $n\,k$ is 3,800 lbs., the compression in BE; $o\,p$ is 2,150 lbs., the compression in EC; $p\,n$ is 3,330 lbs., the compression in ED; $p\,r$ is 1,650 lbs., the tension in the king-post CD; $r\,q$ is 2,660 lbs., the compression in AC.

Collecting the strains obtained by the two diagrams, we have—

Tension in tie-beam	$= 4{,}500 + 4{,}400 = 8{,}900$	lbs.
,, ,, king-post	$= 2{,}900 + 1{,}650 = 4{,}550$	,,
Compression in rafter	$= 5{,}250 + 3{,}800 = 9{,}050$	,,
,, ,, strut	$= 1{,}400 + 3{,}330 = 4{,}730$	,,

We can now, by applying the rules given in works on the strength of materials, find the scantlings required in the case before us. Suppose the framing to be of Memel fir, the safe tensile strain of which is 1,200 lbs. per square inch, then the tie-beam need only have a transverse section of $7\frac{1}{2}$ square inches, and the king-post of $3\frac{3}{4}$ square inches; as, however, each half of the tie-beam has to carry a distributed load of 1,200 lbs., the tie-beam, if made 7 in. deep, must be 2 in. thick. To find the strength of the rafters and struts, we must consider them as long pillars, and use the formula—

$$W = 2{,}500 \times \frac{d^4}{l^2},$$

which is the safe load when d is the diameter in inches and l the length in feet. Putting $l = 6$ feet, we find the rafters must have a scantling of $3\frac{3}{4}$ in. square, and the struts about $3\frac{1}{4}$ in. square. In these dimensions,

however, no allowance is made for cutting away portions of the timber for mortises or for defective portions of the wood; and Tredgold's scantlings for such a roof are as follows:—Tie-beam $9\frac{1}{2} \times 4$, king-post 4×3, rafters 4×4, braces $3\frac{1}{2} \times 2$; in which the strength of the tie-beam is excessive, whilst that of the braces is rather deficient. A better arrangement would be:—Tie-beam 7×3, king-post $2\frac{1}{2} \times 3$, rafters $4\frac{1}{2} \times 3$, braces $3\frac{1}{2} \times 3$.

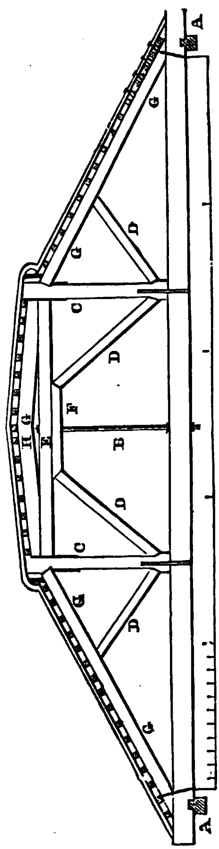


Fig. 15.

	Inch. scantling.
AA, is the tie-beam, 57 feet long, spanning 51 feet clear	14 × 12
CC, Queen-posts	9 × 12
D, Struts	9 × 7
E, Truss beam	10 × 7
F, Straining-piece	6 × 7
G, Principal rafters	10 × 7
HG, A cambered beam for the platform	9 × 7
B, An iron rod, supporting the tie-beam	2 × 2

16. We shall now proceed to examine the arrangement of the timbers in tie-beam roofs of which the span exceeds 50 ft. Fig. 15 represents one truss of the roof of the chapel at Greenwich Hospital, constructed by Mr. S. Wyatt, the span of which is 51 ft. The trusses are 7 ft. apart, and the whole is covered with lead, the boarding being supported by horizontal ledgers, *h,h,* of 6 by 4 ins. This roof contains less timber than most others of the same dimensions. The

parts are all disposed with great judgment. Perhaps the iron rod is unnecessary; but it adds great stiffness to the whole. The iron straps at the rafter feet would have more effect if placed at a greater obliquity. Those at the head of the post are very effective.

The struts (D, F, D) appear to have been inserted in order to stiffen the straining beam (E); but this might have been effected by a king-bolt through the vertex of the rafters HG, which carry the flat over the centre of the roof.

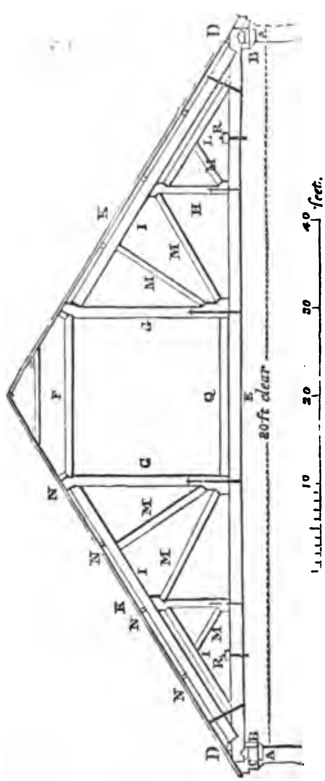


Fig. 16.

	Inch. scantling.
A, is an oak corbel	9 × 5
B, Inner plate	9 × 9
C, Wall plate	8 × 5½
D, Pole plate	7 × 5
E, Tie-beam	15 × 15
F, Straining beam	12 × 9
G, Oak king-post (in the shaft)	9 × 9
H, Oak queen-post (in the shaft)	7 × 9
I, Principal rafters	9 × 9
K, Common ditto	4 × 2½
L, Principal braces	9 and 6 × 9
M, Struts	6 × 9
N, Purlins	7 × 5
Q, Straining sill	5½ × 9

Fig. 16 shows the roof of the Birmingham Theatre, constructed by Mr. George Saunders. The span is 80 ft. clear, and the trusses are 10 ft. apart. The roof is a fine specimen of English carpentry, and is remarkable for its boldness and lightness. The straining sill (Q) gives a firm abutment to the principal struts (M), and the space between the posts being 19½ ft. wide, affords roomy workshops for the carpenters and other workmen connected with a theatre. There is also added a beam (marked R), bolted down to the tie-beams. The intention of this was to prevent the total failure of so bold a trussing, if any of the tie-beams should fail at the end by rot.

It will be observed that we have here a combination of the king and queen post truss, the main feature being a queen-truss, while additional strength is obtained on the two side bays by the introduction of king-trusses.

Fig. 17 represents the roof of Drury Lane Theatre, 80 ft. 3 ins. in the clear, and the trusses 15 ft. apart, constructed by Mr. Edward Grey Saunders.

	Inch. scantling.
A, Tie-beams	10 × 7
B, Rafters	7 × 7
C, King-posts	12 × 7
D, Struts	5 × 7
E, Purlins	9 × 5
G, Pole plates	5 × 5
I, Common rafters	5 × 4
K, Tie-beam to the main truss	15 × 12
L, Posts to ditto	15 × 12
M, Principal braces to ditto	14 and 12 × 12
N, Struts	8 × 12
P, Straining beams	12 × 12

The main beams are strengthened in the middle space with oak trusses 5 in. square.* This was necessary for its width of 32 ft., occupied by the carpenters, painters, &c. The great space between the trusses

* See also Atlas to "Carpentry," Plate IV.

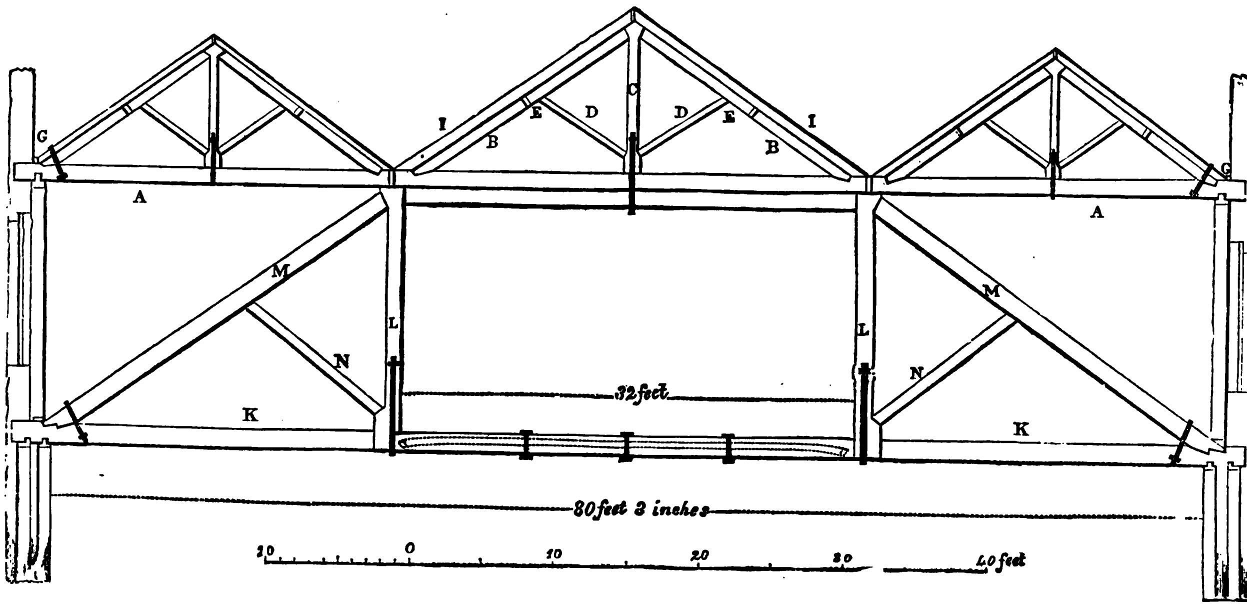

G
A
I
B
E
D
C
D
E
I
B
G
A
M
L
N
K
L
N
M
K
32 feet
80 feet 3 inches
10
0
10
20
30
40 feet

Fig. 17.

affords good store-rooms, dressing-rooms, &c. The main trusses are so framed, that each of them will bear a load of near 300 tons; so it is not likely that they will ever be quarter loaded. The division of the whole into three parts makes the exterior roofings very light. The strains are kept from the walls, and the walls are even firmly bound together by the roof. They also take off the dead weight from the main truss one-third.

17. In order to provide habitable rooms in the roof of a house, the CURB or MANSARD form was introduced (Fig. 18), in which there are four rafters in place of the two used in an ordinary roof, forming an external angle (A) at their junction, the collar-beam (C) serving the double purpose of tying-in the feet of the top rafters, and of carrying the ceiling of the room below. The feet of the lower rafters are also secured to the ends of the floor joists. The lower rafters are generally placed nearly vertical, or inclined at a high angle to the horizon. Some very large roofs of this kind were introduced into the palaces of France during the seventeenth century by the architect Mansard, and have been named after him ever since, the use of such roofs having continued down to the present time in most buildings of importance. Of late years they have made their appearance in buildings erected in England after the style of the French Renaissance or Louis-Quatorze period.

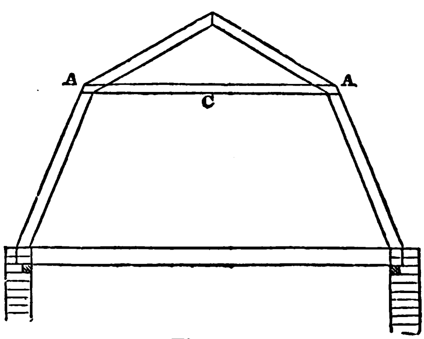


Fig. 18.

When the curb roof is used to cover a wide span, such a truss as that shown below (Fig. 19) may be employed, the common rafters being carried upon it by means of purlins, as previously described(9).

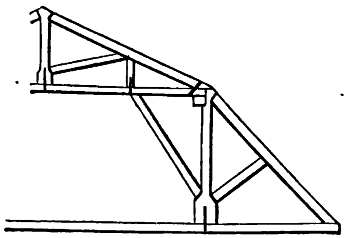


Fig. 19.

The roof over the theatre of the University of Oxford (Fig. 20) may be considered as a curb roof of low pitch, since it has an external angle formed at the junction of the two rafters on each side. The span of this roof is 75 ft., and it will be seen that the framing is a combination of the king and queen truss. The horizontal thrust on the tie-beam is about twice the weight of the roof, and is withstood by an iron strap below the beam which stretches the whole width of the building in the form of a rope, making part of the ornament of the ceiling. This roof is of too low a pitch to be recommended for imitation in roofs of so large a span, as the shrinkage of the timbers is liable to produce settlements, and to put the truss out of shape.

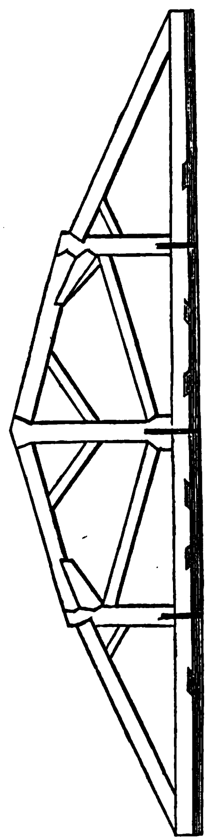


Fig. 20.

18. The Proportions of the Timbers of a Roof depend so much on the design

the framing, that it would be impossible to furnish rules which should apply directly to all cases. Nevertheless, by considering a few combinations, the method that may be adopted will be seen, and consequently may be applied to designs made on other principles than those already shown.

THE KING-POST is intended to support the ceiling, and by means of the braces to support part of the weight of the roof. The weight suspended by the king-post will be proportioned to the span of the roof, and it will also carry half the weight of the tie, the other half being carried by the walls.

Tredgold's rule for the scantling of the king-post is as follows:—Multiply its length in feet by the span in feet; then multiply this product by ·12 for fir, or by ·13 for oak, which will give the area of section of the post in inches; and this area divided by the breadth will give the thickness, or by the thickness will give the breadth.

QUEEN-POSTS AND SUSPENDING PIECES are strained in a similar manner to king-posts, but the load upon them arising from the weight of the tie-beam is only proportional to that part of the length of the tie-beam held up by each suspending piece or queen-post; in queen-posts the part suspended by each is generally one-third the span, as one-third of the weight of the tie is borne directly by the walls.

Tredgold's rule for the scantling of queen-posts is:—Multiply the length in feet of the post by that part of the length of the tie-beam which it supports, also in feet; this product multiplied by ·27 for fir, or by ·32 for oak, will give the area of section of the post in inches; and this area divided by the thickness will give the breadth.

A TIE-BEAM is affected by two strains—the one in the direction of the length from the thrust of the principal rafters; the other is a cross strain from its own weight and that of the ceiling below. In esti-

mating the strength, the thrust of the rafters need not be considered when there is a ceiling to carry, because the beam must in that case be abundantly strong to resist this strain; and when a beam is strained in the direction of the length, it rather increases the strength to resist a cross strain. Therefore the pressure, or the weight supported by the tie-beam, will be proportional to the length of the longest part of it that is unsupported. But there are two cases—one where the weight is merely the weight of the ceiling; the other where there are rooms in the roof, in which case the scantling of the tie-beam must be that of a girder or binder of the same span.

The rule for the scantling of the tie-beam given by Tredgold for the first case is—Divide the length of the longest unsupported part by the cube-root of the breadth; and the quotient multiplied by 1·47 will be the depth required for fir, in inches; or multiplied by 1·52 will give the depth for oak, in inches.

In estimating the strength of PRINCIPAL RAFTERS, we may suppose them to be supported by struts, either at or very near all the points which the purlins rest upon. The pressure on a principal rafter is in the direction of its length, and is in proportion to the magnitude of the roof; but the effect of this pressure does not bear the same proportion to the weight when there is a king-post as when there are queen-posts; therefore the same constant number will not answer for both cases. Tredgold's rule for the scantling is—Multiply the square of the length of the rafters in feet by the span in feet, and divide the product by the cube of the thickness in inches. The depth in inches of a fir beam is found by multiplying this quotient by ·96 where there is a king-post, or by ·155 where there are two queen-posts. The thickness is generally the same as that of the king or queen post and of the tie-beam.

A Straining Beam is the horizontal piece between the heads of the queen-posts, and the pressure is in the direction of its length, and is the same as that sustained by the rafters. In order that this beam may be the strongest possible, its depth should be to its thickness as 10 is to 7.

Tredgold's rule for the scantling of the straining beam is—Multiply the square root of the span in feet by the length of the beam in feet, and extract the square root of the product. Multiply the root by ·9 for fir, which will give the depth in inches. To find the thickness, multiply the depth by ·7.

That part of a roof which is supported by a Strut or Brace is easily ascertained from the design, but the effect of the load must depend on the position of the brace. When it is square from the back of the rafter, the strain upon it will be the least; and when it has the same inclination as the roof, the same strain will be thrown on the lower part of the principal rafter as is borne by the strut. If a piece intended for a brace, a principal rafter, or a straining beam, be curved, the convex side should be placed upwards.

To determine the scantling, Tredgold gives the following rule:—Multiply the square root of the length supported in feet by the length of the brace or strut in feet; and the square root of the product multiplied by ·8, for fir, will give the depth in inches; and this multiplied by ·6 will give the breadth in inches.

The stress upon Purlins is proportional to the distance they are apart; and the weight being uniformly diffused, the stiffness is reciprocally as the cube of the length. Purlins should always be notched down upon the principal rafters, and should be put on in as long lengths as they can be conveniently got, as the strength is nearly doubled by this means. The old

method of framing the purlins into the principal rafters, not only renders the purlins weaker, but also wounds the principal rafter, and consequently renders it necessary to make the rafters stronger.

There is no part of a roof so liable to fail as the purlins; indeed there are few cases where they have not sunk considerably, and in some instances so much as to deform the external appearance of the roof. Weak purlins might be much strengthened by bracing them—a practice which was once very common among the builders in this country. Blocks should be spiked to the upper face of the rafter, against which the side of the purlin can rest so as to be prevented from twisting.

Tredgold's rule for the scantling of a purlin is—Multiply the cube of its length in feet by the distance of the purlins apart in feet; and the fourth root of the product for fir will give the depth in inches; or multiplied by 1·04 the depth for oak.; and the depth multiplied by ·6 will give the breadth

COMMON RAFTERS are uniformly loaded, and the breadth need not be more than from 2 ins. to 2½ ins. Foreign fir of straight grain makes the best common rafters and purlins, because it is not so subject to warp and twist with the heat of roofs in summer as oak; much, however, depends on the quality of the timber; oak from old trees often stands very well.

Where the covering is of Welsh slate the rule for the scantling of rafters given by Tredgold is—Divide the length of bearing in feet by the cube root of the breadth in inches; and the quotient multiplied by ·72 for fir, or by ·74 for oak, gives the depth in inches.

19. In all timber framing for roofs a certain quantity of IRON is necessary for the purpose of holding the several pieces firmly together.

Thus the tie-beam must be upheld to the king or queen post by means of a wrought-iron strap passed under the tie-beam, and held on to the post by turning in the ends and passing a bolt horizontally through the strap and foot of the post. It is better, however, to cut off the foot of the post a little short of the top of the tie-beam, and then pull up the latter to it by driving iron wedges through the upper part of the strap until the tie-beam is just touching the post, giving a slight curvature or *camber* to the beam itself. Tredgold recommends that the strength of the strap should vary according to the length s of the unsupported part of the tie-beam, and should be of the following dimensions:—

When $s = 10$ ft.,	the strap to be	1 in.	wide by	$\frac{3}{16}$-in.	thick.
,, 15 ft.,	,, ,,	$1\frac{1}{2}$ in.	,,	$\frac{1}{4}$ in.	,,
,, 20 ft.,	,, ,,	2 ins.	,,	$\frac{1}{4}$ in.	,,

A strap of wrought iron is also placed round the heel of the principal rafter, for which it forms an abutment in case the end of the tie-beam should fail. If this strap is put too upright it will, instead of forming an abutment, become quite loose when the roof settles; and as it is intended to prevent the rafter foot from sliding along the tie-beam, an oblique position will be the most efficient. Very frequently this strap is not made sufficiently oblique, and is sometimes made almost square with the beam. When this is the case, it not only keeps the foot of the rafter from flying out, but it binds it down, so that the rafter acts as a powerful lever, whose fulcrum is the inner angle of the shoulder, and then the strap never fails to cripple the rafter at the point. All this can be prevented only by making the strap very long and very oblique, and by making its outer end (the stirrup part) square with its length, and making a notch in the rafter foot

to receive it. It cannot now cripple the rafter, for it will rise along with it, turning round the bolt at its inner end. We have been thus particular on this joint, because it is here that the ultimate strain of the whole roof is exerted, and its situation will not allow the excavation necessary for making it a good mortise and tenon. These straps should be about the same strength as those used for the king-post, and in bolting them on they should be drawn quite tight. Sometimes a bolt with head and screw-nut is used instead of a strap to secure the rafter to the tie-beam, in which case it should be placed at right-angles to the back of the rafter, and be provided with plates of iron or washers to prevent it from bruising the timber.

All ironwork should be well coated with paint or other preservative before being used in timber framing.

Cast-iron shoes are sometimes laid upon the walls to receive the ends of the tie-beam, and to protect them from the damp and decay to which they are liable if allowed to rest on the brick or stonework of the walls.

The mode of fixing the straps and bolts, as recommended by Tredgold, is shown on Plates 16 and 17 of Atlas to "Carpentry."

20. In the framings which we have hitherto considered the whole of the outward thrust upon the walls is counteracted by the horizontal tie-beam, which also serves to keep the walls in position, especially when these are not very thick. But there is one great defect in a tie-beam roof, namely, that all the space above it is lost in the room which it covers, and this in public halls, churches, or theatres, is a very considerable amount, and involves the necessity of carrying up the side walls to a greater height than would otherwise be required if the tie-beam could be dispensed with. Various methods have at different times been adopted

for constructing roofs without a tie-beam, and in which the outward thrust on the walls is reduced to a minimum. None of these, however, are suitable unless there is a considerable amount of strength in the walls themselves to resist the thrust of which there must always be more or less, but we shall now proceed to consider those methods which may be considered the most successful. The simplest of these is shown in Fig. 21, in which the two *braces* DA, DC are substituted for the horizontal tie-beam, being held up by the king-post DB, an iron strap in the form of a ⊥ being bolted on the three timbers at their junction.

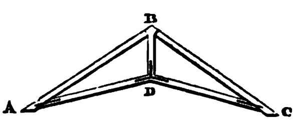


Fig. 21.

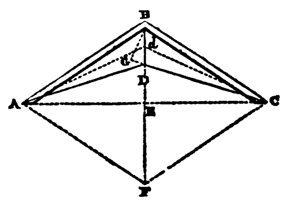


Fig. 22.

The following are the remarks of Professor Robison on the mechanical principles involved in this kind of roof:—Suppose two rafters AB, BC (Fig. 22) movable about the joint B, and resting on the top of the walls. If the string BD be suspended from B, and the two strings DA, DC, be fastened to the feet of the rafters, and if these strings be incapable of extension, it is plain that all thrust is removed from the walls as effectually as by a common tie-beam. And by shortening BD to B *d*, we gain a greater inside height, and more room for an arched ceiling. Now if we substitute a king-post BD (Fig. 21) and two stretchers or hammer-beams DA, DC, for the other strings, and connect them firmly by means of iron straps, we obtain our purpose.

Let us compare this roof with a tie-beam roof in

point of strain and strength. Recur to Fig. 22, and complete the parallelogram ABCF, and draw the diagonals AC, BF, crossing in E. Draw BG perpendicular to CD. We have seen (14) that the weight of the roof (which we may call W) is to the horizontal thrust at C as BF to EC; and if we express this thrust by T, we have $T = \frac{W \times EC}{BF}$. We may at present consider BC as a lever movable round the joint B, and pulled at C in the direction EC by the horizontal thrust, and held back by the string pulling in the direction CD. Suppose that the forces in the directions EC and CD are in equilibrium, and let us find the force S by which the string CD is strained. These forces must (by the property of the lever) be inversely as the perpendiculars drawn from the centre of motion on the lines of their direction. Therefore $BG : BE = T : S$, and $S = T \times \frac{BE}{BG}, = W \times \frac{BE.EC}{BF.BG}$.

Therefore the strain upon each of the ties DA and DC is always greater than the horizontal thrust or the strain on a simple tie-beam. This would be no great inconvenience, because the smallest dimensions that we could give to these ties, so as to procure sufficient fixtures to the adjoining pieces, are always sufficient to withstand this strain. But although the same may be said of the iron straps which make the ultimate connections, there is always some hazard of imperfect work, cracks, or flaws, which are not perceived. We can judge with tolerable certainty of the soundness of a piece of timber, but cannot say so much of a piece of iron. Moreover, there is a prodigious strain excited on the king-post, when BG is very short in comparison of BE, namely, the force compounded of the two strains S and S on the ties DA and DC.

But there is another defect from which the straight tie-beam is entirely free. All roofs settle a little. When this roof settles, and the points B and D descend, the legs BA, BC, must spread further out, and thus a pressure outwards is excited on the walls. It is seldom, therefore, that this kind of roof can be executed in this simple form, and other contrivances are necessary for counteracting this supervening action on the walls. Fig. 23 is a good example of the kind, and is executed with great success in the circus or equestrian theatre in Edinburgh, the width being 60 ft. The pieces EF and ED help to take off some of the weight, and by their greater uprightness they exert a smaller thrust on the walls. The beam D *d* is also a sort of truss beam, having something of the same effect.

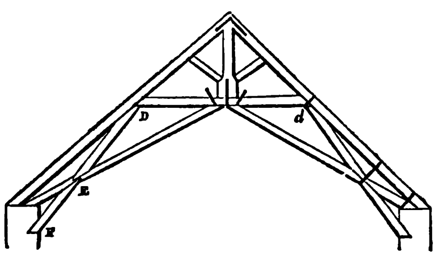


Fig. 23.

21. Another very ingenious mode of obtaining height within a roof and getting rid of the troublesome tie-beam, is that invented by Delorme, and frequently employed in covering large public halls, Fig. 24. In this method a series of curved ribs are placed so that their lower ends stand upon a curb at the base, and the

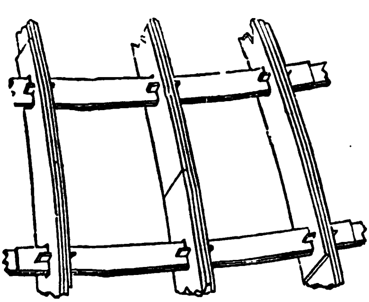


Fig. 24.

upper ends meet at the top, diagonal struts being introduced between them. These ribs are formed of planks put together in thicknesses, with the joints crossed, and well bolted together; there should be at least three thicknesses in each rib, not bent, but applied flat together in a vertical plane, and their edges cut to the proper curvature; the layers of the ribs may be held together without bolts, by merely the horizontal rings or purlins, which pass through a mortise hole in the middle, and have themselves a slit into which a wooden key is driven on each side of the rib, as shown in the figure.

A good example of this method is seen in the roof of the Pantheon in Oxford Street, London, of which the span is 37 ft. The ribs consist of three thicknesses of planks, 7 ft. 6 ins. long, bolted together, and having iron end abutments in every part. The centre thickness is 5 ins. of teak, the side thicknesses of fir are each $2\frac{1}{2}$ ins., the maximum depth being 16 ins., and the whole forming a complete semicircle. Any thrust arising from these ribs is counteracted by the semi-trusses which form the lean-to roofs of the side galleries. For details of the construction of this roof the reader is referred to Tredgold's "Carpentry."*

This kind of roof was, however, especially designed by Delorme for the purpose of constructing TIMBER DOMES, and the following are the scantlings which he gives for domes of different sizes:—For a dome 24 ft. diameter the ribs are to be 8 ins. × 1 in.; for 36 ft. diameter the ribs are 10 ins. × $1\frac{1}{2}$ in.; for 60 ft. they are 13 ins. × 2 ins.; for 90 ft. they are 13 ins. × $2\frac{1}{2}$ ins.; and for 108 ft. the scantling is 13 ins. × 3 ins. The ribs of domes are formed in two thicknesses, and are placed about 2 ft. apart at the base. The rafters are notched upon them for receiving the boarding, and also hori-

* Crosby Lockwood & Co.

zontal ties are notched on the inside, which gives a great degree of stiffness to the whole. In domes of great magnitude it is desirable to place an iron hoop around it, about one-third of the height, to prevent the dome from bursting outwards. In the dome of the Church of the Salute at Venice, which is 80 ft. in diameter, the ribs are 96 in number, each being in four thicknesses, making altogether 5½ ins., with a depth of 8½ ins. The iron hoop is 4½ ins. wide and ½ in. thick, placed at one-third the height of the dome. In this case the lantern is carried by an internal dome of brick; but in cases where the timber dome has to carry the whole weight of the lantern, the framing may be made strong enough by using two ribs, one below the other, with braces between, and tied together by radial pieces across from rib to rib (see Fig. 6, Plate 7, of the Atlas to "Carpentry").

A dome, the ribs of which are constructed of trussed frames (Fig. 25), was erected over the Register Office at Edinburgh, having a span of 50 ft., and a depth of 4½ ft.

Fig. 25.

One of the largest domes ever constructed of timber was that of the old Halle au Blé (Corn-market) in Paris, 200 ft. in diameter, which was the invention of an intelligent carpenter, the Sieur Moulineau. He was not by any means a man of science, but had much more mechanical knowledge than artisans usually have, and was convinced that a very thin shell of timber might not only be so shaped as to be nearly in equilibrium, but that if

hooped or firmly connected horizontally, it would have all the stiffness that was necessary; and he presented his project to the magistracy of Paris. The grandeur of it pleased them, but they doubted of its possibility. Being a great public work, they prevailed on the Academy of Sciences to consider it. The members, who were competent judges, were instantly struck with the justness of Moulineau's principles, and were astonished that a thing so plain had not been long familiar to every house-carpenter. It quickly became a universal topic of conversation, dispute, and cabal in the polite circles of Paris. But the Academy having given a very favourable report of their opinion, the project was immediately carried into execution, and soon completed. It was burnt down in the year 1802, and is now replaced by one of smaller dimensions, erected with ribs of iron covered with sheets of copper.

The construction of this dome was the simplest thing that can be imagined. The circular ribs which composed it consisted of planks 9 ft. long, 13 ins. broad, and 3 ins. thick; and each rib consisted of three of these planks bolted together in such a manner that no two joints met. A rib was begun, for instance, with a plank 3 ft. long standing between one of 6 ft. and another of 9 ft., and this was continued to the head of it. No machinery was necessary for carrying up such small pieces, and the whole went up like a piece of bricklayer's work. At various distances these ribs were connected horizontally by purlins and iron straps, which made so many hoops to the whole. When the work had reached such a height that the distance of the ribs was two-thirds of the original distance, every third rib was discontinued, and the space was left open and glazed. When carried so much higher that the distance of the ribs was one-third of the original distance.

every second rib (now consisting of two ribs very near each other) was in like manner discontinued, and the void was glazed. A little above this the heads of the ribs were framed into a circular ring of timber, which formed a wide opening in the middle; over which was a glazed canopy or umbrella, with an opening between it and the dome for allowing the heated air to get out.

22. The HAMMER-BEAM ROOF, which was adopted by the builders of the fourteenth and fifteenth centuries, is also another mode by which height is obtained in a room and the tie-beam got rid of, the construction of which is shown by Fig. 26.

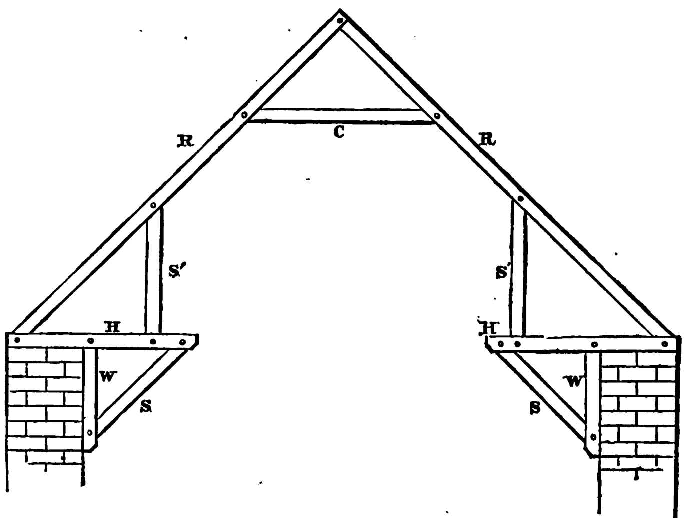


Fig. 26.

In this kind of roof we may suppose that the feet of the rafters are first prevented from spreading by being framed into a tie-beam; the middle part of the tie-beam is afterwards cut away, and the remaining parts (marked H) are called *hammer-beams*. To prevent these beams

from thrusting outwards, a diagonal strut (marked S) is framed into the inner end of each, and also into a vertical *wall-piece* (W), which is itself framed into the under side of the hammer-beam. A vertical strut (marked S′) is also placed between the rafter and the end of the hammer-beam. By this means a considerable amount of the thrust of the rafters is thrown vertically down the walls. There will, however, always remain sufficient horizontal thrust to push out the walls, if they are not built very strong, or supported by external buttresses.

The principle of the construction of these roofs is based on the property of the triangle, that whilst the lengths of the sides remain the same, the angles which they make with each other are unchangeable; and in this case all the pieces of timber are arranged to form the sides of triangles, so that the joints are all rendered immovable. Thus what might at first sight have the appearance of being a load upon the roof is, in fact, its strength and stability.

The method of determining the strains on the several parts of a hammer-beam roof is investigated at page 122 of "The Science of Building," Second Edition.*

The most remarkable specimen of hammer-beam roof, as well as the largest and most magnificent, is that of Westminster Hall (Fig. 27).

The angle of the roof is formed at a pitch of 52°, the length of the rafters being about three-fourths of the entire span, which is 68 ft., the length being 238 ft., and divided into twelve bays by thirteen principals. The cutting off the tie-beams—which, crossing from wall to wall in common roofs, restrain all lateral expansion—and forming them into hammer-beams, was the first circumstance peculiar to this construction. To provide against lateral pressure, we find

* Crosby Lockwood & Co.

trusses, or principals, as they are technically called, raised at the distances of about 18 ft. throughout the whole length of the building. The trusses abut against the solid parts of the walls between the windows, which are strengthened in those parts by arch buttresses on the outside. Every truss comprehends one large arch, springing from corbels of stone, which project from the walls at 21 ft. below the base line of the roof, and at nearly the same height from the floor. The ribs forming this arch are framed at its crown into a collar-beam, which connects the rafters in the middle of their length. A small arch is turned within this large one, springing from the inner ends of the hammer-beams; and supported by two brackets, or half-arches, issuing from the springers of the main arch. By this construction of the trusses, each one acts like an arch; and by placing their springers so far below the top of the walls, a more firm abutment is obtained; subordinate timbers co-operate to transfer the weight and pressure of intermediate parts upon the principals; and thus the whole structure reposes in perfect security, after more than four centuries from its first erection. Great rigidity appears to be given to the main arch by the two small arches in combination with the hammer-beam, thereby

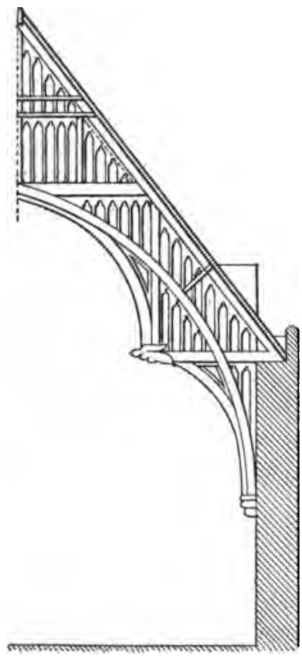


Fig. 27.

giving it the form of a truss, and preventing any great horizontal thrust from falling upon the walls.

The roof over the great hall at Hampton Court somewhat resembles that of Westminster in the principles of its construction, although in this case there is no main arch, but an arched rib springs from the inner end of each hammer-beam, and a curved strut below carries the thrust well down the wall, and rests on a stone corbel 12 ft. below the hammer-beam; while a straight strut, or brace, is fixed from the end of the hammer-beam next the wall to the vertex of the arch, so as to form a kind of truss of two triangles, by which great stiffness is given to the framing. The span of this roof is 40 ft., and the pitch 50°, the hammer-beams projecting 10 ft. from the walls. It has two collar-beams, the upper one being fixed to the top of the main rafters, and a flat termination being given to the roof, so as to resemble a curb roof.

The roof of the hall at Eltham, in Kent, has hammer-beams projecting one-fifth of the span, which is 36 ft., the pitch being 50°; in this case the arched struts are so flat that they throw the load on the top of the wall instead of bringing it down low upon it.

Many excellent examples of the hammer-beam roof of moderate span are to be found over the ancient churches of Norfolk and Suffolk. In some cases we find that there is no proper collar-beam, the arched strut itself acting in that capacity, as in the roof of Wymondham Church, Norfolk, which has a span of 24 ft., and a pitch of only 38°. In this roof, each of the hammer-beams projects one-fifth of the span, and from their inner ends spring curved ribs meeting at a point about 2 ft. below the vertex of the rafters, and a curved strut also springs from a corbel 6 ft. below the hammer-beam. We find another roof of similar construction at

Trunch Church, Norfolk, with a span of 19 ft., and a pitch of 48°; here the hammer-beams have a very great projection, and the scantling of the principal timbers is 10″ × 10″. There is also the roof of St. Stephen's, Norwich, with a span of 17 ft., and a pitch of 45°, the scantling of whose principals is 12″ × 11″; also that of Palgrave Church, Suffolk, with a span of 20 ft., and a pitch of 43°, where the hammer-beam projects only 3 ft., and the scantling of the principals is 13″ × 10″. Another similar roof, having a span of 25 ft., is seen at South Creak Church, Norfolk; and one of 29 ft. span, with a pitch of 40°, at Worstead, Norfolk, where the hammer-beams project only 4½ ft. from the wall.

Among the hammer-beam roofs which are provided with straight collar-beams may be mentioned that of Freslingfield Church, Suffolk, whose span is 20 ft., and pitch 39°; here the hammer-beams project 3½ ft., and are very much tilted upwards, and a flat arch springs from their inner ends to the centre of the collar; the scantling of the collar-beam is 14″ × 8″, and that of the other principal timbers, 10″ × 8″. Another example of this class of roof is at the church of Capel St. Mary, Suffolk, where the collar is at two-thirds the height from the hammer-beam, which latter projects only 2 ft. from the wall, and forms the base of an arched rib rising to the middle of the collar; the span of this roof is 18 ft., and the pitch 46°, the scantling of the principal beams being 10″ × 8″.

In some cases we find a second hammer-beam introduced half-way between the one which rests on the wall and the collar-beam; such a roof does not gain anything in strength from the second hammer-beam with its brace and strut; which, moreover, add considerably to its load upon the walls. One of the largest

of these double hammer-beam roofs is that over Knapton Church, Norfolk, of which the span is 30 ft. and the pitch 38°; the scantling of the collar-beam being 15″×7″, and of the other timbers 12″×10″. Other double hammer-beam roofs of small span are to be seen over the churches of Woolpit, Bacton, and Grundisburgh, in Suffolk.

In all these ancient roofs we find the timbers fastened together by means of wooden pins driven through the tenons; no ironwork in the shape of bolts or straps being employed in their construction.

Among hammer-beam roofs which have been erected in recent times we may mention that over the hall of the Manchester Assize Courts, designed by Mr. Waterhouse, of which the span is 48½ ft. and the pitch 55°, the distance apart of the principals being 14 ft. The hammer-beams in this case extend about half-way across the hall, and are supported below by curved struts springing from stone corbels half-way between the floor and the eaves; while from the inner ends of the hammer-beams spring arched ribs meeting at the centre of the collar-beam, which is placed half-way up the principal rafters, and is held up by a king-post truss. Longitudinal timber trusses are supported on the inner ends of the hammer-beams, and run the whole length of the building.

23. Among simple COLLAR-BEAM ROOFS with arched braces erected in modern times, that designed by Mr. E. M. Barry for the Leeds Grammar School, in 1859, is a good example. In this case the tie-beam is dispensed with, a collar-beam being placed half-way up the principals.

Figs. 28, 29, 30, and 31 represent the roof over the principal school-room, which consists of 6 bays 16 ft. wide, each bay containing a dormer. The main trusses

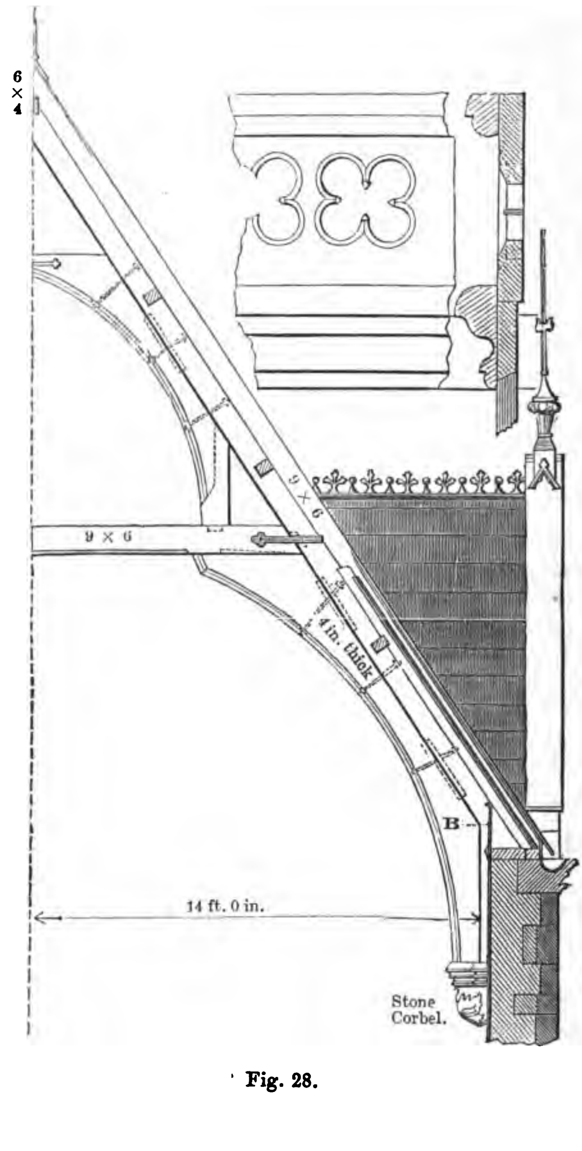
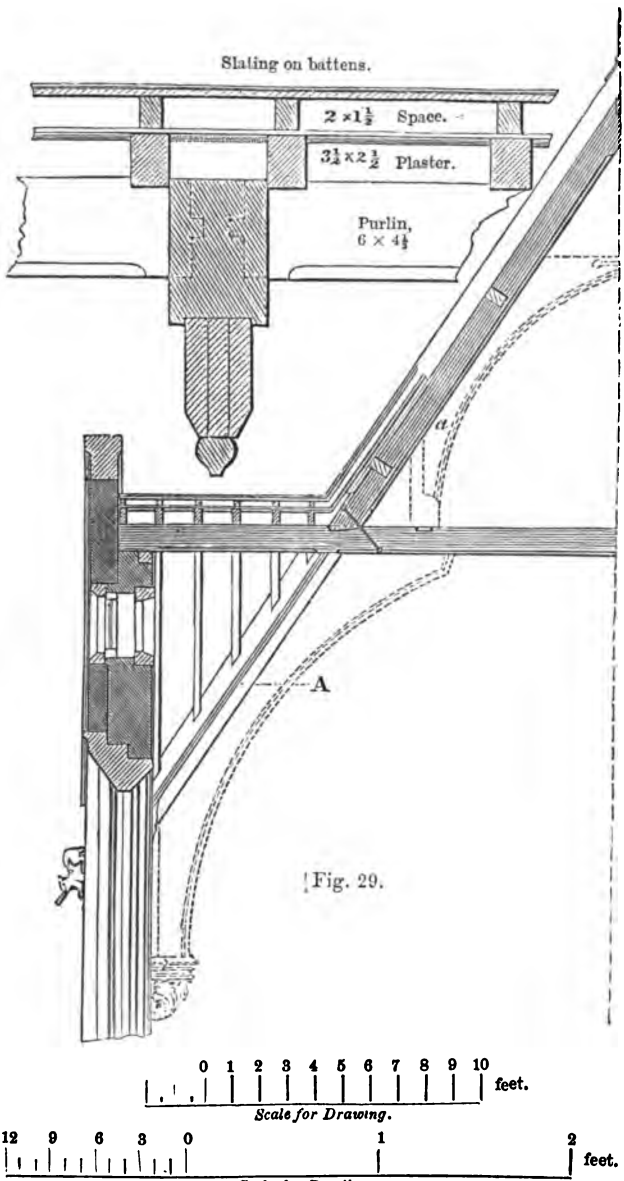


Fig. 28.



Slating on battens.
2 × 1½ Space.
3½ × 2½ Plaster.
Purlin,
6 × 4½
a
A
0 1 2 3 4 5 6 7 8 9 10 feet.
Scale for Drawing.
12 9 6 3 0 1 2 feet.
Scale for Details.

Fig. 29.

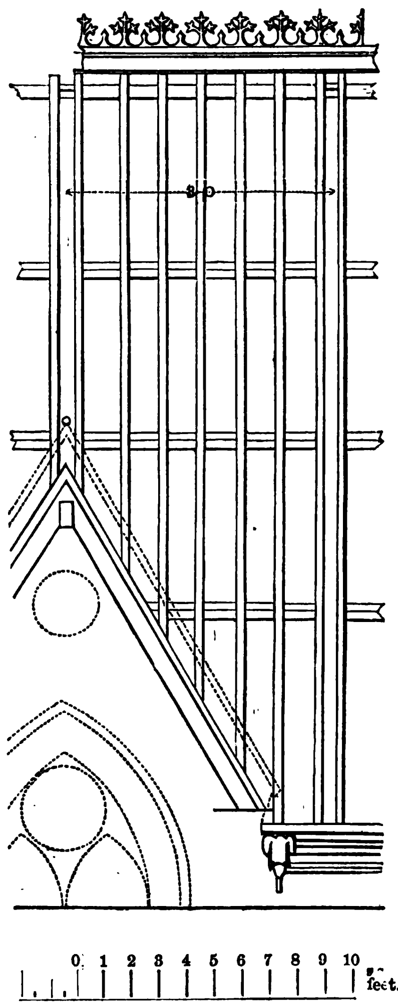


Fig. 30.

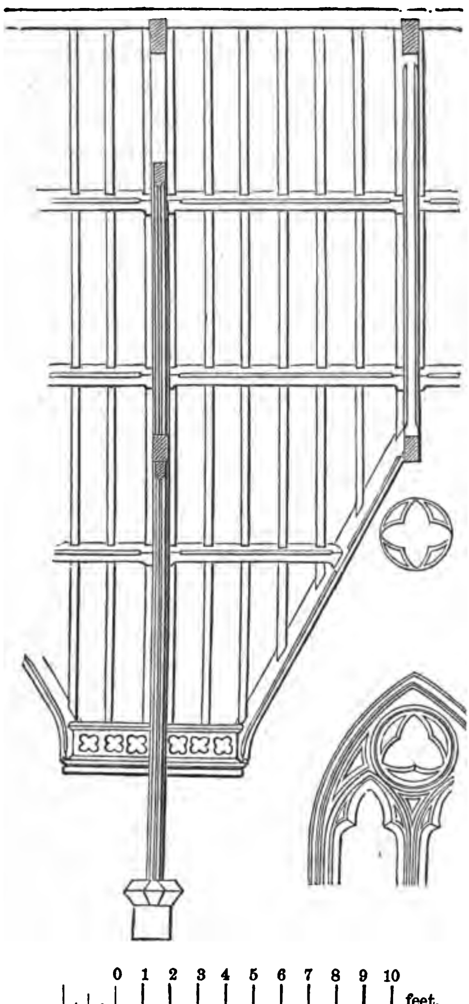


Fig. 31.

have arched ribs at A and B (Figs. 28, 29), which spring from stone corbels and serve to stiffen the part of the principals below the collar; and in the centre of each bay is an intermediate truss without arched ribs. The collar-beam of the intermediate truss forms the ridge of the roofs of dormers, and is in one continuous piece from wall to wall. The wood is Memel fir, and is stained and varnished. The span and scantlings of timbers are as follows:—

Span	28 feet.
Width of bay	16 feet.
Principal rafters	9 ins. × 6 ins.
Common rafters	$3\frac{1}{2}$ ins. × $2\frac{1}{2}$ ins.
Fillets under battens	2 ins. × $1\frac{1}{2}$ ins.
Arched ribs	4 ins. thick.
Ridge	6 ins. × 4 ins.
Collar	9 ins. × 6 ins.

24. Before concluding the subject of timber roofs we shall give extracts from what Professor Robison has written on the theory of strains to which the various pieces of the framing of a roof are subjected.

The force with which the materials of our edifices, roofs, floors, machines, and framings of every kind, resist being broken or crushed, or pulled asunder, is, immediately or ultimately, the cohesion of their particles. When a weight hangs by a rope it tends either immediately to break all the fibres, overcoming the cohesion among the particles of each, or it tends to pull one parcel of them from among the rest with which they are joined. The union of the fibres is brought about by some kind of gluten, or by twisting, which causes them to bind each other so hard, that any one will break rather than come out, so much is it withheld by friction. The ultimate resistance is therefore the cohesion of the fibre; the force or strength of all fibrous materials, such as timber, is exerted in much the same manner. The fibres are either broken or

pulled out from among the rest. Metals, stone, glass, and the like, resist being pulled asunder by the simple cohesion of their parts.

The force which is necessary for breaking a rope or wire is a proper measure of its strength. In like manner, the force necessary for tearing directly asunder any rod of wood or metal, breaking all its fibres, or tearing them from among each other, is a proper measure of the united strength of all these fibres. And it is the simplest strain to which they can be exposed, being just equal to the sum of the forces necessary for breaking or disengaging each fibre. And, if the body is not of a fibrous structure, which is the case with metals, stones, glass, and many other substances, this force is still equal to the simple sum of the cohesive forces of each particle which is separated by the fracture. Let us distinguish this mode of exertion of the cohesion of the body by the name of its ABSOLUTE STRENGTH.

When solid bodies are, on the contrary, exposed to great compression, they can resist only to a certain degree. A piece of clay or lead will be squeezed out; a piece of freestone will be crushed to powder; a beam of wood will be crippled, swelling out in the middle, and its fibres lose their mutual cohesion, after which it is easily crushed by the load. A notion may be formed of the manner in which these strains are resisted by conceiving a cylindrical pipe filled with small shot, well shaken together, so that each spherule is lying in the closest manner possible, that is, in contact with six others in the same vertical plane (this being the position in which the shot will take the least room). Thus each touches the rest in six points. Now suppose them all united, in these six points only, by some cement. This assemblage will stick together and form a cylin-

drical pillar, which may be taken out of its mould. Suppose this pillar standing upright, and loaded above. The supports arising from the cement act obliquely, and the load tends either to force them asunder laterally, or to make them slide on each other: either of these things happening, the whole is crushed to pieces. The resistance of fibrous materials to such a strain is a little more intricate, but may be explained in a way very similar.

A piece of matter of any kind may also be destroyed by wrenching or twisting it. We can easily form a notion of its resistance to this kind of strain, by considering what would happen to the cylinder of small shot if treated in this way.

And lastly, a beam, or a bar of metal, or a piece of stone or other matter, may be broken transversely. This will happen to a rafter or joist supported at the ends when overloaded, or to a beam having one end stuck fast in a wall and a load laid on its projecting part. This is the strain to which materials are most commonly exposed in roofs; and, unfortunately, it is the strain which they are the least able to bear; or rather it is the manner of application which causes an external force to excite the greatest possible immediate strain on the particles. It is against this that the carpenter must chiefly guard, avoiding it when in his power, and, in every case, diminishing it as much as possible. It is necessary to give the reader a clear notion of the great weakness of materials in relation to this transverse strain. But we shall do nothing more, referring him to the works on the STRENGTH OF MATERIALS.

25. Let ACDB, Fig. 32, represent the side of a beam projecting horizontally from a wall in which it is firmly fixed, and let it be loaded with a weight W

appended to its extremity. This tends to break it; and the least reflection will convince any person that if the beam is equally strong throughout, it will break in the line CD, even with the surface of the wall. It will open at D, while C will serve as a sort of joint or fulcrum round which it will turn. The cross section through the line CD is, for this reason, called the *section of fracture*, and the horizontal line, drawn through C on its under surface, is called the *axis of fracture*. The fracture is made by tearing asunder the fibres, such as DE or FG. Let us suppose a real joint at C, and that the beam is really sawn through along CD, and that in place of its natural fibres threads are substituted all over the section of fracture. The weight now tends to break these threads; and it is our business to find the force necessary for this purpose.

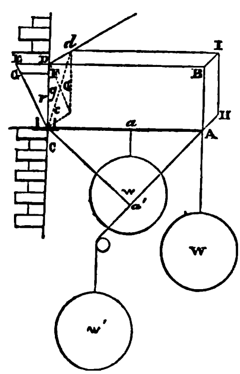


Fig. 32.

It is evident that DCA may be considered as a bent lever, of which C is the fulcrum. If f be the force which will just balance the cohesion of a single thread when hung on it so that the smallest addition will break it, we may find the weight which will be sufficient for this purpose when hung on at A, by saying, AC : CD $= f : \phi$, and ϕ will be the weight which will just break the thread, by hanging ϕ by the point A. This gives us $\phi = f \times \frac{\text{CD}}{\text{CA}}$. If the weight be hung on at a, the force just sufficient for breaking the same thread will be $= f \times \frac{\text{CD}}{\text{C}a}$. In like manner the force ϕ,

which must be hung on at A in order to break an equally strong or an equally resisting fibre at F, must be $= f \times \frac{\text{CF}}{\text{CA}}$. And so on for all the rest.

If we suppose all the fibres to exert equal resistances at the instant of fracture, we know, from the simplest elements of Mechanics, that the resistance of all the particles in the line CD, each acting equally in its own place, is the same as if all the individual resistances were united in the middle point g. Now this total resistance is the resistance or strength f of each particle, multiplied by the number of particles. This number may be expressed by the line CD, because we have no reason to suppose that they are at unequal distances. Therefore, in comparing different sections together, the number of particles in each are as the sections themselves. Therefore DC may represent the number of particles in the line DC. Let us call this line the depth of the beam, and express it by the symbol d. And since we are at present treating of roofs whose rafters and other parts are commonly of uniform breadth, let us call AH or BI the breadth of the beam, and express it by b, and let CA be called its length, l. We may now express the strength of the whole line CD by $f \times d$, and we may suppose it all concentrated in the middle point g. Its mechanical energy, therefore, by which it resists the energy of the weight w, applied at the distance l, is f. CD. Cg, while the moment of w is w. CA. We must therefore have f. CD. Cg. $= w$. CA, or $f. d. \frac{1}{2} d = w. l$, and $f. d : w = l : \frac{1}{2} d$, or $f. d : w = 2\ l : d$. That is, twice the length of the beam is to its depth as the absolute strength of one of its vertical planes to its relative strength, or its power of resisting this transverse fracture.

It is evident that what has been now demonstrated

of the resistance exerted in the line CD, is equally true of every line parallel to CD in the thickness or breadth of the beam. The absolute strength of the whole section of fracture is properly represented by $f. d. b$, and we still have $2\ l : d = f\ d\ b : w$; or twice the length of the beam is to its depth as the absolute strength to the relative strength. Suppose the beam 12 ft. long and 1 ft. deep; then whatever is its absolute strength, the 24th part of this will break it if hung at its extremity.

But even this is too favourable a statement; all the fibres are supposed to resist alike in the instant of fracture. But this is not true. At the instant that the fibre at D breaks, it is stretched to the utmost, and is exerting its whole force. But at this instant the fibre at g is not so much stretched, and it is not then exerting its utmost force. If we suppose the extension of the fibres to be as their distance from C, and the actual exertion of each to be as their extensions, it may easily be shown that the whole resistance is the same as if the full force of all the fibres were united at a point r distant from C by two-thirds of CD. In this case we must say, that the absolute strength is to the relative strength as three times the length to the depth; so that the beam is weaker than by the former statement in the proportion of two to three.

Even this is more strength than experiment justifies; and we can see an evident reason for it. When the beam is strained, not only are the upper fibres stretched, but the lower fibres are compressed. This is very distinctly seen, if we attempt to break a piece of cork cut into the shape of a beam: this being the case, C is not the centre of fracture. There is some point c which lies between the fibres which are stretched and those that are compressed. This fibre is neither

stretched nor squeezed; and this point is the real centre of fracture: and the lever by which a fibre D resists, is not DC, but a shorter one D *c*; and the energy of the whole resistances must be less than by the second statement. Till we know the proportion between the dilatability and compressibility of the parts, and the relation between the dilatations of the fibres and the resistances which they exert in this state of dilatation, we cannot positively say where the point *c* is situated, nor what is the sum of the actual resistances, or the point where their action may be supposed concentrated. The firmer woods, such as oak and chestnut, may be supposed to be but slightly compressible; we know that willow and other soft woods are very compressible. These last must therefore be weaker: for it is evident, that the fibres which are in a state of compression do not resist the fracture. It is well known, that a beam of willow may be cut through from C to *g* without weakening it in the least, if the cut be filled up by a wedge of hard wood stuck in.

We can only say, that very sound oak and red fir have the centre of effort so situated, that the absolute strength is to the relative strength in a proportion not less than that of three and a half times the length of the beam to its depth. A square inch of sound oak will carry about 8,000 lbs. If this bar be firmly fixed in a wall, and project about 12 in., and be loaded at the extremity with 200 lbs. it will be broken. It will just bear 190, its relative strength being $\frac{1}{42}$ of its absolute strength; and this is the case only with the finest pieces, so placed that their annual plates or layers are in a vertical position. A larger log is not so strong transversely, because its plates lie in various directions round the heart.

These observations are enough to give us a distinct

notion of the vast diminution of the strength of timber when the strain is across it: and we see the justice of the maxim, that the carpenter, in framing roofs, should avoid as much as possible the exposing his timbers to transverse strains. But this cannot be avoided in all cases. Nay, the ultimate strain, arising from the very nature of a roof, is transverse. The rafters must carry their own weight, and this tends to break them across: an oak beam a foot deep will not carry its own weight if it project more than 60 ft. Beside this, the rafters must carry the lead, tiling, or slates. We must therefore consider this transverse strain a little more particularly, so far as to know what strain will be upon any part by any unavoidable load, laid on either at that or at any other.

26. We have hitherto supposed that the beam had one of its ends fixed in a wall, and that it was loaded at the other end. This is not a usual arrangement, and was taken merely as affording a simple application of the mechanical principles. It is much more usual to have the beam supported at the ends, and loaded in the middle. Let the beam FEGH, Fig. 33, rest on the props E and G, and be loaded at its middle point C with a weight W. It is required to determine the strain at the section CD? It is plain that the beam will receive the same support, and suffer the same strain, if, instead of the blocks E and G, we substitute the ropes F f e, H h g, going over the pulleys f and h, and loaded with proper weights e and g. The weight e is equal to the support given by the block E; and g is equal to the support given by G. The sum of e and

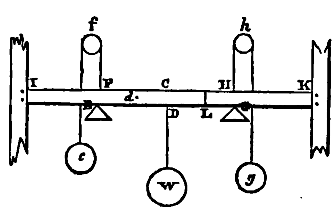


Fig. 33.

g is equal to W; and, on whatever point W is hung, the weights *e* and *g* are to W in the proportion of DG and DE to GE. Now, in this state of things, it appears that the strain on the section CD arises immediately from the upward action of the ropes F *f* and H *h*, or the upward pressures of the blocks E and G; and that the office of the weight W is to oblige the beam to oppose this strain. Things are in the same state in respect of strain as if a block were substituted at D for the weight W, and the weights *e* and *g* were hung on at E and G; only the directions will be opposite. The beam tends to break in the section CD, because the ropes pull it upwards at E and G, while a weight W holds it down at C. It tends to open at D, and C becomes the centre of fracture. The strain therefore is the same as if the half ED were fixed in the wall, and a weight equal to *g*, that is, to the half of W, were hung on at G.*

Hence we conclude, that a beam supported at both ends, but not fixed there, and loaded in the middle, will carry *four times* as much weight as it can carry at its extremity, when the other extremity is fast in a wall.

The strain occasioned at any point L by a weight W, hung on at any other point D, is $= W \times \frac{DE}{EG} \times LG$. For EG is to ED as W is to the pressure occasioned at G. This would be balanced by some weight *g* acting over the pulley *h*; and this tends to break the beam at L, by acting on the lever GL. The pressure at G is $W.\ \frac{DE}{EG}$, and therefore the strain at L is $W.\ \frac{DE}{EG}.\ LG$.

* Gregory's "Mathematics for Practical Men." Crosby Lockwood & Co.

In like manner, the strain occasioned at the point D by the weight W hung on there, is $W \times \frac{DE}{EG} \times DG$; which is therefore equal to $\frac{1}{2}$ W, when D is the middle point.

Hence we see, that the general strain on the beam arising from one weight, is proportional to the rectangle of the parts of the beam (for $\frac{W.DE.DG}{EG}$ is as DE.DG), and is greatest when the load is laid on the middle of the beam.

We also see, that the strain at L, by a load at D, is equal to the strain at D by the same load at L. And the strain at L, from a load at D, is to the strain by the same load at L as DE to LE. These are all very obvious corollaries; and they sufficiently inform us concerning the strains which are produced on any part of the timber by a load laid on any other part.

If we now suppose the beam to be fixed at the two ends, that is, firmly framed, or held down by blocks at I and K, placed beyond E and G, or framed into posts, it will carry twice as much as when its ends were free. For suppose it sawn through at CD; the weight W hung on there will be just sufficient to break it at E and G. Now restore the connection of the section CD, it will require another weight W to break it there at the same time.

Therefore, when a rafter, or any piece of timber, is firmly connected with the three fixed points G, E, I, it will bear a greater load between any two of them than if its connection with the remote point were removed; and if it be fastened in four points, G, E, I, K, it will be twice as strong in the middle part as without the two remote connections.

One is apt to expect from this that the joist of a floor will be much strengthened by being firmly built in the wall. It is a little strengthened; but the hold which can thus be given it is much too short to be of any sensible service; and it tends greatly to shatter the wall, because, when it is bent down by a load, it forces up the wall with the moment of a long lever. Judicious builders therefore take care not to bind the joists tight in the wall. But when the joists of adjoining rooms lie in the same direction, it is a great advantage to make them of one piece. They are then twice as strong as when made in two lengths.

27: It is easy to deduce from these premises the strain on any point which arises from the weight of the beam itself, or from any load which is uniformly diffused over the whole or any part. We may always consider the whole of the weight which is thus uniformly diffused over any part as united in the middle point of that part; and if the load is not uniformly diffused, we may still suppose it united at its centre of gravity. Thus, to know the strain at L arising from the weight of the whole beam, we may suppose the whole weight accumulated in its middle point D. Also the strain at L, arising from the weight of the part ED, is the same as if this weight were accumulated in the middle point d of ED; and it is the same as if half the weight of ED were hung on at D. For the real strain at L is the upward pressure at G, acting by the lever GL. Now calling e the weight of the part DE: this upward pressure will be, $\frac{e \times d\mathrm{E}}{\mathrm{EG}}$, or $\frac{\frac{1}{2} e \times \mathrm{DE}}{\mathrm{EG}}$.

Therefore the strain on the middle of a beam, arising from its own weight, or from any uniform load, is the weight of the beam or its load $\times \frac{\mathrm{ED}}{\mathrm{EG}} \times \mathrm{DG}$; that is, half

the weight of the beam or load multiplied or acting by the lever DG; for $\frac{ED}{EG}$ is $\frac{1}{2}$.

Also the strain at L, arising from the weight of the beam, or the uniform load, is $\frac{1}{2}$ the weight of the beam or load acting by the lever LG. It is therefore proportional to LG, and is greatest of all at D. Therefore a beam of uniform strength throughout, uniformly loaded, will break in the middle.

28. It is of importance to know the relation between the strains arising from the weights of the beams, or from any uniformly diffused load, and the relative strength. Leaving out every circumstance but what depends on the dimensions of the beam, it is shown by writers on the strength of materials that the relative strength is in the proportion of the breadth and the square of the depth directly, and the length inversely. (See "The Science of Building.")

Now, to consider first the strain arising from the weight of the beam itself, it is evident that this weight increases in the same proportion with the depth, the breadth, and the length of the beam. Therefore its power of resisting this strain must be as its depth directly, and the square of its length inversely. To consider this in a more popular manner, it is plain that the increase of breadth makes no change in the power of resisting the actual strain, because the load and the absolute strength increase in the same proportion with the breadth. But by increasing the depth, we increase the resisting section in the same proportion, and therefore the number of resisting fibres and the absolute strength: but we also increase the weight in the same proportion. This makes a compensation, and the relative strength is yet the same. But by increasing the depth, we have not only

increased the absolute strength, but also its mechanical energy: for the resistance to fracture is the same as if the full strength of each fibre was exerted at the point which we called the centre of effort; and we showed, that the distance of this from the under side of the beam was a certain portion (a half, a third, a fourth, &c.) of the whole depth of the beam. This distance is the arm of the lever by which the cohesion of the wood may be supposed to act. Therefore this arm of the lever, and consequently the energy of the resistance, increases in the proportion of the depth of the beam, and this remains uncompensated by any increase of the strain. On the whole, therefore, the power of the beam to sustain its own weight increases in the proportion of its depth. But, on the other hand, the power of withstanding a given strain applied at its extremity, or to any aliquot part of its length, is diminished as the length increases, or is inversely as the length; and the strain arising from the weight of the beam also increases as the length. Therefore the power of resisting the strain actually exerted on it by the weight of the beam, is inversely as the square of the length. On the whole, therefore, the power of a beam to carry its own weight, varies in the proportion of its depth directly, and the square of its length inversely.

As this strain is frequently a considerable part of the whole, it is proper to consider it apart, and then to reckon only on what remains for the support of any extraneous load.

In the next place, the power of a beam to carry any load which is uniformly diffused over its length, must be inversely as the square of the length: for the power of withstanding *any* strain applied to an aliquot part of the length (which is the case here, because the

load may be conceived as accumulated at its centre of gravity, the middle point of the beam) is inversely as the length; and the *actual* strain is as the length, and therefore its momentum is as the square of the length. Therefore the power of a beam to carry a weight uniformly diffused over it, is inversely as the square of the length. *N.B.* It is here understood, that the uniform load is of some determined quantity for every foot of the length, so that a beam of double length carries a double load.

29. We have hitherto supposed that the forces which tend to break a beam transversely, are acting in a direction perpendicular to the beam. This is always the case in level floors loaded in any manner; but in roofs, the action of the load tending to break the rafters is oblique, because gravity always acts in vertical lines. It may also frequently happen, that a beam is strained by a force acting obliquely. This modification of the strain is easily discussed. Suppose that the external force, which is measured by the weight W in Fig. 32, acts in the direction A w' instead of AW. Draw C $\acute{a}$ perpendicular to A w. Then the moment of this external force is not to be measured by W × AC, but by W × $\acute{a}$ C. The strain therefore by which the fibres in the section of fracture DC are torn asunder, is diminished in the proportion of CA to C $\acute{a}$.

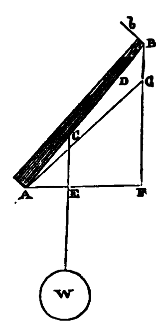


Fig. 34.

To apply this to our purpose in the most familiar manner, let AB, Fig. 34, be an oblique rafter of a building, loaded with a weight W suspended to any point C, and thereby occasioning a strain in some part D. We have already seen, that the immediate cause of the strain

on D is the reaction of the support which is given to the point B. The rafter may at present be considered as a lever, supported at A, and pulled down by the line CW. This occasions a pressure on B, and the support acts in the opposite direction to the action of the lever, that is, in the direction B *b*, perpendicular to BA. This tends to break the beam in every part. The pressure exerted at B is, $\frac{W \times AE}{AB}$, AE being a horizontal line. Therefore the strain at D will be $\frac{W \times AE}{AB} \times BD$. Had the beam been lying horizontally, the strain at D, from the weight W suspended at C, would have been $\frac{W.AC}{AB} \times BD$; it is therefore diminished in the proportion of AC to AE.

It is evident that this law of diminution of the strain is the same whether the strain arises from a load on any part of the rafter, or from the weight of the rafter itself, or from any load uniformly diffused over its length, provided only that these loads act in vertical lines.

30. We can now compare the strength of roofs which have different elevations. Supposing the width of the building to be given, and that the weight of a square yard of covering is also given. Then, because the load on the rafter will increase in the same proportion with its length, the load on the slant side BA of the roof will be to the load of a similar covering on the half AF of the flat roof, of the same width, as AB to AF. But the transverse action of any load on AB, by which it tends to break it, is to that of the same load on AF as AF to AB. The transverse strain, therefore, is the same on both, the increase of real load on

AB being compensated by the obliquity of its action. But the strengths of beams to resist equal strains, applied to similar points, or uniformly diffused over them, are inversely as their lengths, because the moment or energy of the strain is proportional to the length. Therefore the power of AB to withstand the strain to which it is really exposed, is to the power of AF to resist its strain, as AF to AB. If, therefore, a rafter AG of a certain scantling is just able to carry the roofing laid on it, a rafter AB of the same scantling, but more elevated, will be too weak in the proportion of AG to AB. Therefore steeper roofs require stouter rafters, in order that they may be equally able to carry a roofing of equal weight per square yard. To be equally strong, they must be made broader, or placed nearer to each other, in the proportion of their greater length, or they must be made deeper in the subduplicate proportion of their length.

31. We proceed, in the next place, to consider the other strains to which the parts of roofs are exposed in consequence of the support which they mutually give each other, and the pressures, or *thrusts*, which they exert on each other, and on the walls or piers of the building.

Let a beam or piece of timber AB (Fig. 35) be suspended by two lines AC, BD ; or let it be supported by two props AE, BF, which are perfectly movable round their remote extremities E, F, or let it rest on the two polished planes KAH, LBM. Moreover, let G be the centre of gravity of the beam, and let GN be a line through the centre of gravity, perpendicular to the horizon. The beam will not be in equilibrium unless the vertical line GN either passes through P, the point in which the directions of the two lines AC, BD, or the directions of the two props EA, FB, or the perpendi-

culars to the two planes KAH, LBM, intersect each other, or is parallel to these directions. For the supports given by the lines or props are unquestionably exerted in the direction of their lengths; and it is well known in Mechanics that the supports given by planes are exerted in a direction perpendicular to those planes in the points of contact; and we know that the weight of the beam acts in the same manner as if it were all accumulated in its centre of gravity G, and that it acts in the direction GN perpendicular to the horizon. Moreover, when a body is in equilibrium between three forces, they are acting in one plane, and their directions are either parallel or they pass through one point.

Fig. 35.

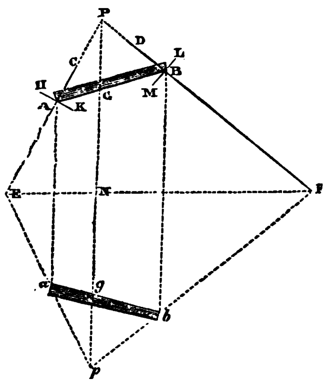


Fig. 36.

The support given to the beam is therefore the same as if it were suspended by two lines which are attached to the single point P. We may also infer, that the points of suspension C, D, the points of support E, F, the points of contact A, B, and the centre of gravity G, are all in one vertical plane.

When this position of the beam is disturbed by any external force, there must either be a motion of the points A and B round the centres of suspension C and D, or of the props round these points of support E and F, or a sliding of the ends of the beam along the polished planes GH and IK; and in consequence of these motions the centre of gravity G will go out of its place, and the vertical line GN will no longer pass

through the point where the directions of the supports intersect each other. If the centre of gravity rises by this motion, the body will have a tendency to recover its former position, and it will require force to keep it away from it. In this case the equilibrium may be said to be *stable*, or the body to have *stability*. But if the centre of gravity descends when the body is moved from the position of equilibrium, it will tend to move still farther; and so far will it be from recovering its former position, that it will now fall. This equilibrium may be called a *tottering equilibrium*. These accidents depend on the situations of the points A, B, C, D, E, F; and they may be determined by considering the subject geometrically. It does not much interest us at present; it is rarely that the equilibrium of suspension is tottering, or that of props is stable. It is evident, that if the beam were suspended by lines from the point P, it would have stability, for it would swing like a pendulum round P, and therefore would always tend towards the position of equilibrium. The intersection of the lines of support would still be at P, and the vertical line drawn through the centre of gravity, when in any other situation, would be on that side of P towards which this centre has been moved. Therefore, by the rules of pendulous bodies, it tends to come back. This would be more remarkably the case if the points of suspension C and D were on the same side of the point P with the points of attachment A and B; for in this case the new point of intersection of the lines of support would shift to the opposite side, and be still farther from the vertical line through the new position of the centre of gravity. But if the point of suspension and of attachment are on opposite sides of P, the new point of intersection may shift to the same side with the centre of gravity, and lie beyond the vertical

line; in this case the equilibrium is tottering. It is easy to perceive, too, that if the equilibrium of suspension from the points C and D be stable; the equilibrium on the props AE and BF must be tottering. It is not necessary for our present purpose to engage more particularly in this discussion.

It is plain that, with respect to the mere momentary equilibrium, there is no difference in the support by threads, or props, or planes, and we may substitute the one for the other. We shall find this substitution extremely useful, because we easily conceive distinct notions of the support of a body by strings.

Observe further, that if the whole figure be inverted, and strings be substituted for props, and props for strings, the equilibrium will still obtain: for by comparing Fig. 35 with Fig. 36, we see that the vertical line through the centre of gravity will pass through the intersection of the two strings or props; and this is all that is necessary for the equilibrium: only it must be observed in the substitution of props for threads, and of threads for props, that if it be done without inverting the whole figure, a stable equilibrium becomes a tottering one, and *vice versâ*.

This is a most useful proposition, especially to the artisan, and enables him to make a practical use of problems which the greatest mechanical geniuses have found no easy task to solve. An instance will show the extent and utility of it. Suppose it were required to make a Mansard or curb-roof whose width is AB (Fig. 37), and consisting of the four equal rafters AC, CD, DE, EB. There can be no doubt but that its

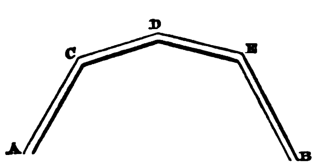


Fig. 37.

best form is that which will put all the parts in equilibrium, so that no ties or stays may be necessary for opposing the unbalanced thrust of any part of it. Make a chain *a c d e b* (Fig. 38) of four equal pieces, loosely connected by pin-joints, round which the parts are perfectly movable. Suspend this from two pins *a*, *b*, fixed in a horizontal line. This chain or festoon will arrange itself in such a form that its parts are in equilibrium. Then we know that if the figure be inverted, it will compose the frame or truss of a curb-roof

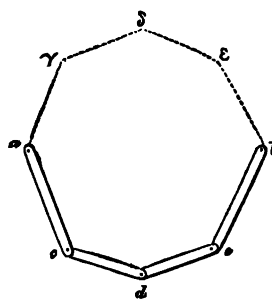


Fig. 38.

$a\ \gamma\ \delta\ \epsilon\ b$, which is also in equilibrium, the thrusts of the pieces balancing each other in the same manner that the mutual pulls of the hanging festoon *a c d e b* did. If the proportion of the height $\frac{1}{2}\ d\ \delta$ to the width *a b* is not such as pleases, let the pins *a*, *b*, be placed nearer or more distant, till a proportion between the width and height is obtained which pleases, and then make the figure ACDEB (Fig. 37) similar to it. It is evident that this proposition will apply in the same manner to the determination of the form of an arch of a bridge; but this is not a proper place for a further discussion.

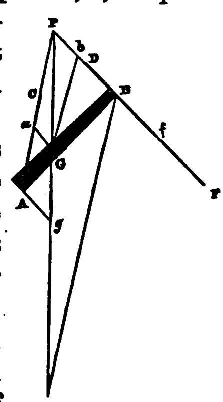


Fig. 39.

We are now able to compute all the thrusts and other pressures which are exerted by the parts of a roof on each other and on the walls. Let AB (Fig. 39) be a beam standing anyhow obliquely,

and G its centre of gravity. Let us suppose that the ends of it are supported in any directions AC, BD, by strings, props, or planes. Let these directions meet in the point P of the vertical line PG, passing through its centre of gravity. Through G draw lines G *a*, G *b*, parallel to PB, PA. Then

$$\left.\begin{array}{l}\text{The weight of the beam}\\ \text{The pressure or thrust at A}\\ \text{The pressure at B}\end{array}\right\}\text{ are proportional to }\left\{\begin{array}{l}PG\\ P\,a\\ P\,b.\end{array}\right.$$

For, when a body is in equilibrium between three forces, these forces are proportional to the sides of a triangle which have their directions.

In like manner, if A *g* be drawn parallel to P *b*, we shall have

$$\left.\begin{array}{l}\text{Weight of the beam}\\ \text{Thrust on A}\\ \text{Thrust on B}\end{array}\right\}\text{ proportional to }\left\{\begin{array}{l}P\,g\\ PA\\ A\,g.\end{array}\right.$$

Or, drawing B γ parallel to P *a*,

$$\left.\begin{array}{l}\text{Weight of beam}\\ \text{Thrust at A}\\ \text{Thrust at B}\end{array}\right\}\text{ are proportional to }\left\{\begin{array}{l}P\,\gamma\\ B\,\gamma\\ PB.\end{array}\right.$$

It cannot be disputed that, if strength alone be considered, the proper form of a roof is that which puts the whole in equilibrium, so that it would remain in that shape although all the joints were perfectly loose or flexible. If it has any other shape, additional ties or braces are necessary for preserving it, and the parts are unnecessarily strained. When this equilibrium is obtained, the rafters which compose the roof are all acting on each other in the direction of their lengths; and by this action, combined with their weights, they sustain no strain but that of compression, the strain of all others that they are the most able to resist. We may consider them as so many inflexible lines having their weights accumulated in their centres of gravity

But it will allow an easier investigation of the subject, if we suppose the weights to be at the joints equal to the real vertical pressures which are exerted on these points. These are very easily computed; for it is plain that the weight of the beam AB (Fig. 39) is to the part of this weight that is supported at B as AB to AG. Therefore, if W represent the weight of the beam, the vertical pressure at B will be $W \times \frac{AG}{AB}$, and the vertical pressure at A will be $W \times \frac{BG}{AB}$. In like manner, the prop BF being considered as another beam, and f as its centre of gravity, and w as its weight, a part of this weight, equal to $w \times \frac{fF}{BF}$, is supported at B, and the whole vertical pressure at B is $W \times \frac{AG}{AB} + w \times \frac{fF}{BF}$. And thus we greatly simplify the construction of the mutual thrusts of roof frames. We need hardly observe, that although these pressures by which the parts of a frame support each other in opposition to the vertical action of gravity, are always exerted in the direction of the pieces, they may be resolved into pressures acting in any other direction which may engage our attention.

We need not repeat that it is always a desirable thing to form a truss for a roof in such a manner that it shall be in equilibrium. When this is done, the whole force of the struts and braces which are added to it is employed in preserving this form, and no part is expended in unnecessary strains. For we must now observe, that the equilibrium of which we have been treating is always of that kind which we call the tot-

tering, and the roof requires stays, braces, or hanging timbers, to give it stiffness or keep it in shape. We have also said enough to enable any reader acquainted with the most elementary geometry and mechanics, to compute the transverse strains and the thrusts to which the component parts of all roofs are exposed.

Thus have we given an elementary, but scientific, account of this important part of the art of carpentry. The sagacity of the carpenter is not enough without science for perfecting the art. For when he knows how much a particular piece will yield to compression in one case, science will tell him what will be the compression of the same piece in another very different case. Thus he learns how far it will now yield, and then he proportions the parts to each other, so that when all have yielded according to their strains, the whole is of the shape he wished to produce, and every joint is in a state of firmness.

PART II.—ROOFS OF IRON.

32. The employment of iron as a material for the framework of roof trusses has only been introduced in recent times, and may indeed be said to have come in with railways, the station roofs of which are usually of this material. In the old timber roofs erected four or five centuries ago we do not find that iron was employed even for the purpose of connecting the several parts of the framework together. But upon the introduction of the trussed roof with horizontal tie-beam, iron bolts and straps were found indispensable, and a vertical suspension bolt was occasionally added to hold up the tie-beam or other horizontal piece, when the length was considerable. In modern times, the king and queen post of wood have often been superseded by a wrought-iron bolt having a cast-iron head into which the principals are tenoned. Where the roof was quite open to the space below, and the tie-beam had no ceiling to support, a tie-rod of wrought iron was often used in place of the tie-beam, being bolted at each end through the feet of the principals.

The advantages which arise from the employment of iron in roof trusses instead of wood are very great, the chief of which we shall now point out. Iron is not

inflammable like wood, and therefore may be used when it is desired to make a building fireproof. Iron does not shrink or get weakened by age as wood does from the drying up of its natural moisture, and can therefore be safely trusted to cover spans for which timber would be quite inadequate. Iron has also a great advantage over wood in being capable of being twisted about into any conceivable shape without loss of strength, so that whereas wood must always be used of rectangular section, we may alter the section of iron in any way we please to suit the purposes to which it is applied. Iron also is vastly stronger, bulk for bulk, than wood, and therefore takes up much less space when equal strength is required; if, however, we compare equal *weights* of iron and wood, we find there is not much to choose between them as regards strength, the specific gravity of iron being about sixteen times that of fir.

There is, however, one great disadvantage under which iron labours, and which to a great extent neutralises its advantages, namely, its tendency to corrode on the surface when exposed to the action of moist air; this corrosion causes the surface to scale off, and thereby reduces its strength by reducing the area of its section or the quantity of material it contains. This circumstance makes it necessary to have the iron much stronger at starting than would otherwise be necessary, in order to allow for reduction of strength by corrosion. Several methods have been suggested and employed for preventing or hindering the corrosion of iron; castings just taken out of the mould are protected for a time by being coated with linseed oil, then smoked over a wood fire, and afterwards dipped in turpentine. Paints of various kinds are employed which are composed either of some metallic oxide or carbonate mixed with oil and

turpentine, or else in the form of a varnish made of a resinous gum, with oil, turpentine, or some kind of spirit. The surface of the metal must be carefully cleansed from all rust before the paint is applied, otherwise the corrosion will go on underneath the paint and throw off the coating. One of the most lasting, but rather expensive, modes of protecting iron is by what is termed "galvanising," or giving it a thin coating of zinc, into a bath of which metal the iron is plunged, the surface of the bath being covered with sal-ammoniac to dissolve the zinc-oxide which forms upon it. The zinc becomes alloyed with the iron, and although itself soon oxidized by contact with air, yet this oxide protects the zinc, and as long as the zinc lasts the iron will be preserved from corrosion. Another process of recent discovery, and which promises to afford permanent protection to the iron, consists in the formation of a coating of the black or magnetic oxide on its surface by passing superheated steam over the metal when red-hot for several hours. The oxide which is thus formed is harder than the metal itself, and adheres firmly to its surface, so that all corrosion is effectually prevented. The expense attending this process is the chief drawback to its general adoption on all structural works of iron, but probably this would be less in the long run than the cost of constantly renewing the ordinary coating of paint.

Wherever it is necessary to cover a wide opening by a roof of one span, iron will always maintain its pre-eminence over wood, as there seems to be hardly any limit to the width of roof that can be constructed with it without the necessity of intermediate supports. The widest roofs are found over railway stations, in which we see iron employed in all manner of ways. The sections of the iron used in roofs are various, the chief

being as follows:—Round iron, flat bar iron, angle-iron (L), tee-iron (T), I-section, cross section (+); to which may be added, in some cases, flat plates of rolled iron varying from $\frac{1}{4}$ in. to $\frac{3}{4}$ in. in thickness.

In the construction of trusses of iron it is essential that great attention should be paid to the proper formation of the *joints* or connections by which the several pieces are united to one another, as everything depends upon their strength, which must always be as great as that of the parts they serve to connect. Where rivets are employed the height of the rivet-head should be at least equal to half its diameter in order to prevent it from being forced off. The diameter of the nut or head of a bolt, or of the head of a rivet, should be twice that of the spindle. When plates with riveted joints are subjected to a tensile strain, the sectional area of all the rivets should be equal to the effective section of the plate.

33. The simplest form which a roof can take is undoubtedly the *flat*, in which there is scarcely any perceptible slope, the inclination being merely sufficient to allow the rain-water to run off freely at one side into a gutter. Such a roof may be constructed of rolled joists of I-section, laid across from wall to wall; the lead, zinc, asphalte, or other covering being laid upon boarding which is fixed to the horizontal joists. If a fireproof roof is required the space between the joists may be filled in with concrete, or by arches of flat tiles or bricks. In this kind of roof the strength of the joists should be the same as for an ordinary floor of equal span. If the joists are placed 4 ft. apart over a span of 12 ft., their scantling may be $5'' \times 4'' \times \frac{3}{8}''$; and for a span of 15 ft. they should have a scantling of $6\frac{1}{4}'' \times 4\frac{3}{4}'' \times \frac{1}{2}''$.

Where wider spans have to be covered, it is better to

place girders of cast or wrought iron across about every 10 ft. length of the building, and on the top or bottom flange of these to lay rolled I-joists as above described, on which the covering is laid. If the span is 20 ft., a beam of cast iron may be used having a ⊥-section, the dimensions at the centre being 1 foot depth, with a width of 8 ins. and thickness of $1\frac{1}{2}$ in. to the bottom flange. For wider spans a wrought-iron beam may be used, formed of I-section with top and bottom *flange* of flat plates riveted by means of L-irons to the *web-plate*, or vertical part; if the span is 30 ft. and the depth 1 foot, the flanges should be 6 ins. wide and the plates $\frac{3}{4}$ in. thick; or if 15 ins. deep the thickness of the metal can be reduced to $\frac{1}{2}$ in.

34. The most usual form of roof, however, is that in which there are two sloping rafters abutting one against the other. In the earlier days of iron roofs the modes of framing resembled those of timber roofs, having the horizontal tie-bar with king or queen bolts, struts, and braces. It was soon found, however, that what was suitable for timber roofs was not the best form for those of iron, as might be expected from the great difference in the character of the two materials. The form of roof given below (Fig. 40) is one that has frequently been adopted where the span was considerable; the principal rafter in this case being usually of T-iron, as it is subjected both to a compressive and a transverse strain. The vertical suspension rods K I, E G, C D, being wholly in tension, are of round or flat bar iron riveted or bolted to the web of the principal rafters. The horizontal tie-bar being chiefly in tension may be also of round or flat bar iron, the latter being preferable where the distances between the feet of the suspension rods is considerable, as in that case there is a transverse strain from its own weight which tends to produce

sagging. The tie-bar is bolted or riveted to the feet of the principal rafters, and is itself upheld in several places by the suspension rods, which are either made to pass through it and secured with a nut and screw below the bar, or else riveted to a socket which is bolted on to the tie-bar. The struts GK, DE, being subjected to a compressive force, are sometimes made of cast iron, either in the form of a hollow cylinder or of the cross section, and made thicker at the middle than the ends in order to resist any transverse strain. Wrought-iron struts or braces are, however, more generally used, the section being either the angle, tee, or cross. In order to carry the slating or other covering, purlins of **I**-section are riveted on the top or to the sides of the principals, and on these again rest the common rafters of **T**-section, which receive the iron laths for the slating to be hung upon, except where close boarding is used and laid directly upon the top of the purlins.

The various strains to which the several parts of this truss are subjected may be determined by the following geometrical process:—Let W be the entire weight which the truss has to sustain; then $\frac{1}{6}$ W is supported *directly* at each of the points K, E, C; $\frac{1}{12}$ W directly at A. Let the dotted lines C *c*, E *b*, K *a*, be equal, and each represent on a scale $\frac{1}{6}$ W. Draw the parallelogram C *c*, making C *d* parallel to BC, and *c d* to

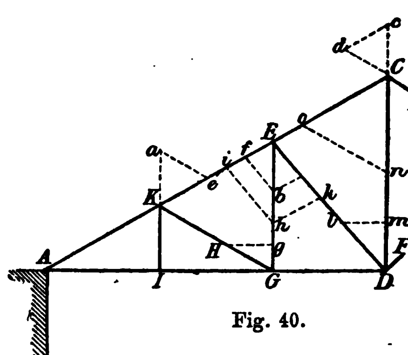


Fig. 40.

AC; then *d c* represents the resolved part of $\frac{1}{6}$ W acting down the rafter AC. Draw the line OX (Fig. 41) parallel to AC, and OY horizontal; and in the line OX take O*p* equal to *c d*. At the point E (Fig. 40) construct the parallelogram E *b*, then E*f* is the resolved part of $\frac{1}{6}$ W acting at E down the rafter AC; *f b* the resolved part down the strut ED. Take *p q* (Fig. 41) equal to E*f*. At K (Fig. 40) draw *a e* parallel to KG, then K *e* is the resolved part of $\frac{1}{6}$ W at K down AC, and *a e* is the resolved part down the strut KG. Make *q r* (Fig. 41) equal to K*e*. Take GH (Fig. 40) equal to *a e*, draw the horizontal H *g*, then G *g*, which is equal to half K *a*, is the vertical strain on EG from the strut KG; take *b h* equal to G *g*, draw *h i* parallel to ED, and *h k* to AC; then *i f* is the resolved part in the direction of AC; take *r s* (Fig. 41) equal to *i f*. The length E *k* (Fig. 40) represents the compression down the strut ED which is conveyed to the point D. Take D *l* equal to E *k*, and draw *l m* horizontal; then twice D *m* represents the vertical strain on CD caused by the two struts ED, FD. Take C *n* equal to twice D *m*, draw *o n* parallel to CB; then C *o* is the compression down AC produced by the rod CD. Take *s* X (Fig. 41) equal to C *o*; then the line OX represents the total compression down the rafter AC on the same scale that C *c* represents $\frac{1}{6}$ W. Draw the vertical XY (Fig. 41), then OY represents the horizontal strain on the tie-rod. One half of the weight of the tie-rod itself is sustained by the vertical rods, so that in this case each suspension rod has to sustain one-tenth of the weight of the tie-rod in addition to its other strains.

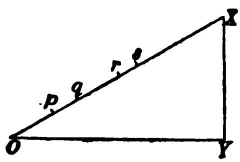


Fig. 41.

In this kind of roof truss the tie-bar need not be

horizontal, but would be equally strong and require less metal if the point D were nearer to the vertex C by one-third or one-fourth of the height of C above A; the length of the struts and ties being in that case shortened.

35. The roofs of the Houses of Parliament, designed by Sir C. Barry, are all constructed of iron; that

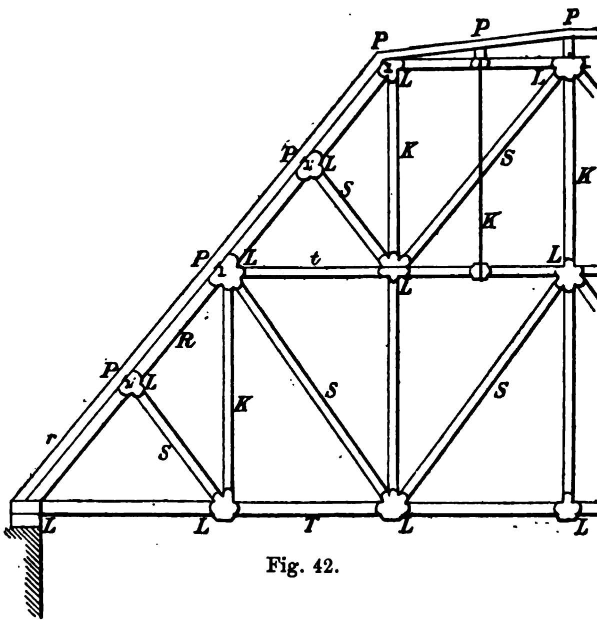


Fig. 42.

which covers the House of Lords has a span of 45 ft., and is rather peculiar in its arrangement; the half truss is shown above (Fig. 42), both wrought and cast iron being employed in its construction. Rolled iron of T-section, whose scantling is 6″ × 3″ × 1″, is used for the principal and common rafters (R and *r*). The two horizontal tie-beams (T and *t*) and the vertical suspension rods (K) are all of flat bar iron, the prin-

cipal tie-beam being 1″ thick and 5″ deep. Cast iron is employed to form the struts (S), which are of cross section; and also for the purlins (P) and the shoes or sockets (L) by which the several parts are united together, the detail of which is shown by Fig. 43.

The principals are placed 7 ft. 6 ins. from centre centre, with two common rafters between; these latter resting in sockets formed on the purlins. The outer

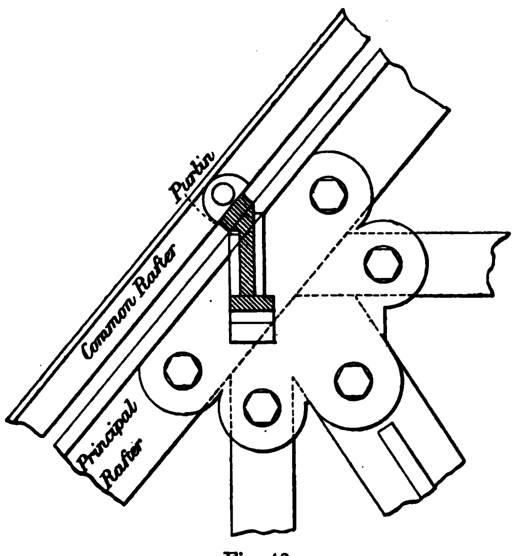


Fig. 43.

covering consists of cast-iron slabs or tiles $\frac{3}{8}$ in. thick and 2 ft. 6 ins. square, the edge on one side being turned up, and on the other formed into a conical roll which covers the turn-up of the next slab, so as to form vertical rolls from the ridge to the gutter every 30 ins. of length. These slabs are held to the rafters by means of two ears which clip over the rafter, and are secured thereto by means of a bolt passed through

it. The slabs are "galvanised," or coated with zinc, to prevent them from corroding. The feet of the principal rafters are fixed by screw bolts and nuts to cast-iron shoes bedded on the wall. The connection of the suspension rods and struts with the tie-beams and principals is effected by means of cast-iron sockets (Fig. 43) through which bolts are passed. The purlins are also bolted to these sockets. The longitudinal connection of the principals is also secured by diagonal cast-iron struts and braces of cross section which are inclined alternately in opposite directions, one being placed between each two contiguous principals and held by bolts and sockets.

36. In roofs which are entirely constructed of iron the horizontal tie-beam is generally replaced by tie-

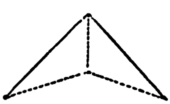


Fig. 44.

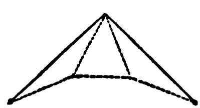


Fig. 45.

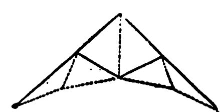


Fig. 46.

rods inclined upwards from the feet of the rafters, as shown above (Figs. 44, 45, 46).

These forms of trusses, although unsuitable for timber-framing, are the most economical where iron is the material employed; the parts which are subjected to tensile strain are in these figures indicated by dotted lines, the strong lines representing those which are either wholly under compression, or which are subjected to a transverse as well as a compressive strain, namely, the rafters and struts. A very simple and effective form of iron roof, which has been largely adopted where the span is not very great, is shown in Fig. 47, in which all the parts are in tension except the rafters AC, BC, and the struts DG, EF. The tie-rods AD, DE, EB are generally in three

separate pieces, united at D and E either by bolts or else by being screwed into a junction-piece; at A and B they are bolted to the feet of the principals, as shown in detail by Figures 50 and 51 (page 85). This

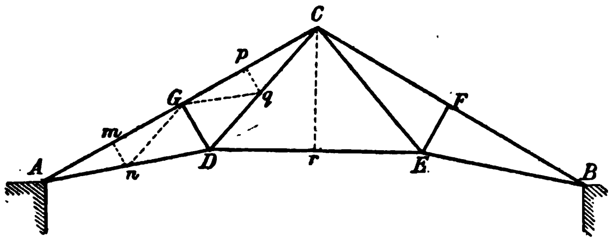


Fig. 47.

form of roof may be safely employed where the span does not exceed 50 ft. The relative strains to which the several parts of a truss of this kind are subjected can be determined geometrically by drawing lines parallel to the directions in which the forces act, as shown by the diagram (Fig. 48). Let W represent the total weight which the truss has to support; then $\frac{1}{8}$ W is *directly* borne by the points A and B; while $\frac{1}{4}$ W is supported at each of the points G, C, and F; and the vertical reaction at A and B is $\frac{1}{2}$ W. Let the vertical line N f

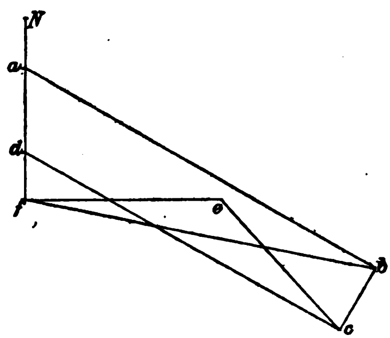


Fig. 48.

represent $\frac{1}{2}$ W on any convenient scale; take N a, f d, each equal (on the same scale) to $\frac{1}{8}$ W. Draw a b, d c parallel to the rafter BC (Fig. 47); f b parallel to the tie-rod BE; b c parallel to the strut EF; c e

parallel to the suspension rod EC; and *fe* parallel to the tie-beam DE. Then, if we measure the lengths of the several lines thus drawn on the above scale, we obtain the strains in the several parts of the roof to which they are respectively parallel. Thus, N*f* represents the reaction at A or B, and equals $\frac{1}{2}$ W,; N *a* is the load supported at A or B, and equals $\frac{1}{8}$ W; *a d* is the vertical pressure at G or F, and equals $\frac{1}{4}$ W; 2 *df* is the vertical pressure at C, and equals $\frac{1}{4}$ W; *a b* is the strain down AG or BF; *d c* the strain down CG or CF; *b c* the compression down EF or GD; *bf* the tension in AD or BE; *e c* the strain along EC or DC; *ef* the tension in DE. We have here supposed that the force of the wind on the surface of the roof is included in the load W; but since the wind presses only upon one side of a roof at a time, the more accurate method is to form a separate diagram for the wind strains as previously described for roofs of timber (Fig. 14, page 20).

This form of roof may be used for larger spans than 50 ft. if the principals are strengthened by the braces and struts shown by the dotted lines *m n*, *p q*, G *n*, G *q*; and the centre tie-rod supported by the king-bolt C *r*, to prevent it from sagging; *m n*, *p q* will be in compression, while G *n*, G *q* will be in tension.

37. A modification of the form of roof which has been described above (36) is shown in Fig. 49, which represents a half truss of the Pimlico Station roof of the Brighton and South Coast Railway. This was designed by Mr. Jacomb Hood, C.E., and completed in 1861; its span is 53 ft., the pitch being 26°, the rafters having an inclination of 1 in 2. The main principals are placed 12 ft. 6 ins. apart, and have T-rafters 4″ × 4″ × $\frac{1}{2}$″, their feet being secured to a "snug" cast upon the gutter. The struts DE, FH are of

wrought-iron piping, fitted with joints at the ends; the ties are of round iron; the tie-rod AH is $1\frac{1}{4}''$ diameter, the part HD is $1\frac{1}{8}''$, and the middle tie DC

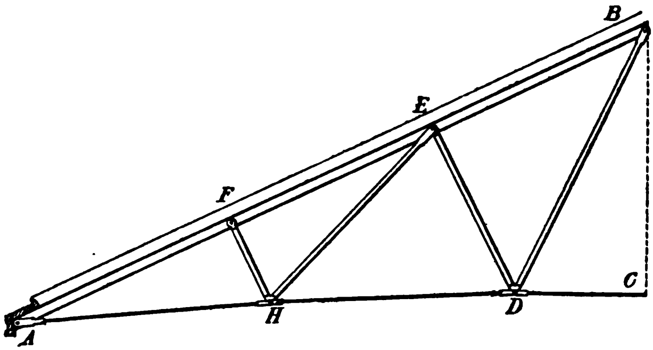
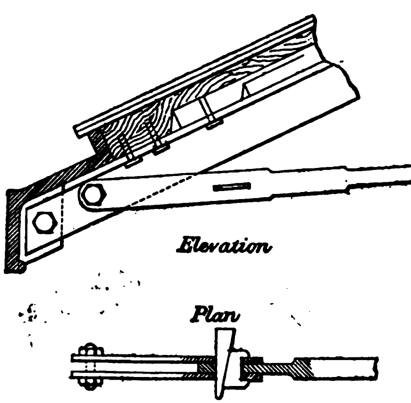


Fig. 49.

is 1 in. The strut HF is $1\frac{1}{2}$ in. tube, and DE is 2 in. tube. The tie EH is $\frac{3}{4}$ in. round iron, and the tie BD is $\frac{7}{8}$ in. diameter The mode of connecting the tie-rods with the feet of the rafters is shown below (Figs. 50, 51); and further details of the construction will be found in Humber's "Record of Modern Engineering" for 1863.* The covering upon the rafters is duchess slating, laid with a lap of 3 ins., and fastened with copper nails to $1\frac{1}{4}$ in. boarding. The boarding is cut into



Figs. 50 and 51.

* Crosby Lockwood & Co

lengths so as to break joint over the main principals only, and is ploughed and tongued with galvanised hoop iron. The boarding is fixed to a deal curb on the backs of the main and intermediate principals, the curb being secured to the rafters by $\frac{3}{8}$ in. square-headed coach screws, 9 ins. apart, on alternate sides of the T-iron. The total weight per square of roofing is a little under a ton. This is a form of roof very commonly employed to cover railway stations of which the span is not very great.

38. Where the span of a roof exceeds 100 ft., it is usual to adopt a form known by the name of the Bowstring truss, in which the rafter is bent in the form of a *bow*, or else in that of a polygon, while the thrust of this arched rib is counteracted by means of a *string* or tie-beam held up at intervals to the bow, or *boom* as it is generally called, by suspension rods, while the boom is stiffened by means of struts and braces, as shown on Fig. 52, which represents the half truss of the roof over Cannon Street Station, London, designed by Mr. Hawkshaw, C.E. This is one of the largest roofs upon the bowstring principle that have been erected, having a clear span of 190 ft. The principals are 33 ft. apart, the centre of the boom is 60 ft. above the springing, and the depth of the truss at the centre is 30 ft. Each boom is 21 ins. deep, and consists of top and bottom plates riveted by four angle irons to a

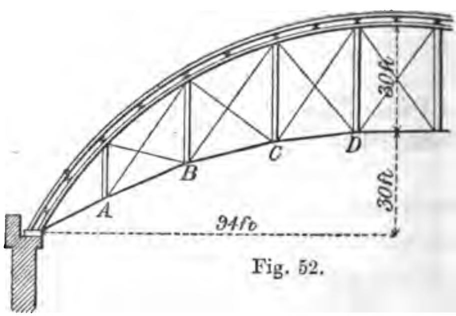


Fig. 52.

full web-plate $\frac{3}{8}$ in. thick, the top and bottom plates being of the same thickness, and 14 ins. in width. The lower end is splayed out at a distance of 10 ft. above the foot until it increases to 4 ft., where it rests on the cast-iron expansion shoe, the intermediate space being filled in with casting. The purlins are placed at intervals of 11 ft., and are full web girders 20 ins. deep, resting on the lower flange of the boom, the top being flush with that of the boom. The web is $\frac{3}{8}$ in. thick, the top flange is $10\frac{3}{8}$ ins. wide and formed of two angle irons; the bottom flange is formed of two angle irons $5'' \times 3'' \times \frac{1}{2}''$. These purlins extend from principal to principal, and carry the covering of the roof, which consists of glass, zinc, and slating. The glass is fixed in wrought-iron sash bars of 3 in. T-iron riveted to the purlins at the upper end, and raised by means of two angle irons at the lower end. The zinc and slate are carried upon $1\frac{1}{2}$ in. boarding, attached to wood packing on the top of the purlins. The diagonal braces or ties are of flat bar iron 6″ wide, and varying in thickness from $\frac{3}{4}$ in. to 1 in.; the tie-rod is of $5\frac{5}{16}$ in. round iron throughout, with rolled heads, by which it is united to the couplings at the junctions (A, B, C, D) with the struts and braces by bolts $4\frac{1}{4}''$ diameter. The vertical struts are of two T-irons kept apart by means of cast-iron distance pieces. The effects of alteration of temperature are provided for by means of rollers, and a cast-iron shoe with circular seat, placed under one end of each principal. The details of the construction of this roof will be found in Humber's "Record of Modern Engineering" for 1866.* In bowstring trusses the vertical pieces are considered as struts subject to compression, while the diagonals are supposed to act only as ties subject to a tensile

* Crosby Lockwood & Co.

strain. This arrangement, however, of the strains may under some circumstances be reversed.

39. The roof over Charing Cross Station, also designed by Mr. Hawkshaw, resembles that of Cannon Street, but the span is less, being 164 ft., the height of the boom at the centre from the springing 45 ft., and the depth of the truss at the middle 20 ft. The principals are placed 35 ft. apart, but there are also two tiers of wrought-iron trussed frames introduced between each pair of principals, which extend parallel thereto from the walls to the eighth purlin on each side; their use is to support the purlins laterally and tie them all firmly together. The boom is 18 ins. deep of I-section, having top and bottom flanges riveted to a plate-iron web, the flanges being of double angle iron $6'' \times 3'' \times \frac{1}{2}''$, and the web $\frac{1}{2}$ in. thick. The lower end of the rib begins to expand at 6 ft. from the foot to a width of 2 ft. at the bearing. The purlins, which extend from principal to principal, are lattice girders 20 ins. deep, resting on the lower flange of the main boom, being attached to it at top and bottom by bolts with slotted holes. They are formed of double angle iron $3\frac{1}{4}'' \times 3'' \times \frac{1}{2}''$, the lattice bars crossing each other, and riveted at the crossing, are $2'' \times \frac{5}{8}''$. To the purlins is attached the roof covering, which consists of glass and zinc; the glass being carried by wrought-iron sash bars of T-section riveted to the purlins; and the zinc upon $1\frac{1}{2}''$ boarding attached to the purlins. Means are provided at one end of each principal for counteracting the effects of expansion and contraction by use of an expansion shoe, for details of which and of the other parts of the structure the reader is referred to Humber's "Record of Engineering" for 1864.* The tie-rods are of round iron, varying in

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diameter from $4\frac{3}{4}$ ins. at the springing to $4\frac{1}{4}$ ins. at the centre. The vertical struts are all of T-iron $6'' \times 3'' \times \frac{1}{2}''$, and the diagonal braces are of flat bar iron $4'' \times \frac{3}{4}''$. The tie-rods are united to each other, as well as to the struts and braces, by being screwed into a cast-iron washer, to which the struts and braces are riveted.

40. The roof of the railway station at Amsterdam is designed on the bowstring principle, but differs essentially in its details from those above described. The span in this case is 120 ft., and the height of the crown above the springing is 30 ft., the depth of the truss at the centre is 13 ft., and the principals are placed 25 ft. apart. The boom is not a continuous curve, but is a segment of a flat polygon, formed of eight cast-iron tubes 8 ins. diameter in straight lengths, with angular connecting pieces; the abutting ends of the tubes and connecting pieces are turned and connected together at each joint by bolts $1\frac{1}{4}''$ diameter. The thickness of metal in the tubes at springing is $\frac{11}{16}$ in., but diminishes to $\frac{9}{16}$ in. at the crown. The ties consist of two flat bars, each $4'' \times \frac{7}{8}''$, and the diagonals, which do not cross each other, are each formed of two flat bars $6'' \times \frac{5}{16}''$, with an angle iron riveted to each bar, and connected together by a wrought-iron plate. The pins and bolts for connecting the diagonals, cast-iron arch, and tie-bars are 3 ins. diameter, and are accurately turned, all the holes through which they pass being bored to fit. The purlins are of cast iron, each in three lengths, 2 ft. 3 ins. deep, and of the lattice form.

41. One of the largest bowstring roofs erected in this country is that over the New Street Station at Birmingham, the span of which is 212 ft.; the principals are 24 ft. apart, and their rise in the centre is 40 ft. above the springing; the depth of the truss at the centre

being 23 ft. The design is very similar to that of the Cannon Street roof (Fig. 52), but the braces and struts are more numerous, there being twelve vertical struts in the Birmingham roof and eight in that of Cannon Street. The boom of each principal is of I-section, formed of a web-plate 15 ins. deep, with two angle irons riveted at top and bottom, forming flanges 13 ins. wide. The tie is of round iron, 4 ins. in diameter, in short lengths screwed into couplings, to which the feet of the struts and braces are attached. The braces are of $\frac{5}{8}$-in. bar iron, varying from 3 ins. to 5 ins. in width. Each strut is composed of four angle irons, placed in the form of a cross, with cast-iron distance pieces to keep them apart, and are swelled out slightly in the middle, by which greater strength is supposed to be obtained. The purlins are of 5-in. die square timber, trussed with a wrought-iron tie rod; they are placed 8 ft. apart. The roof is covered partly with corrugated iron and partly with rough plate glass fixed in wooden sash bars.

42. Another bowstring roof of wide span is that which was erected in 1847 over the Liverpool Station of the London and North-Western Railway, of which the span is 153 ft., the height at the centre 30 ft. above the springing, and the depth of the truss at the middle 12 ft. This roof has an arched boom of rolled iron of I-section 9 ins. deep, having a 10-in. plate $\frac{1}{4}$ in. thick riveted on the top. It is further strengthened near the foot by plates riveted to the sides. There are six vertical tie-rods, with diagonal braces between them; the main tie-bar is 3 ins. diameter. The principals are 21 ft. apart, being fixed at one end, and left free at the other to allow of alteration by change of temperature, the foot being placed upon rollers.

43. The domical roof which covers the elliptical arena of the Albert Hall, at South Kensington, is formed

of bowstring trusses of similar design to those employed on the Cannon Street Station (Fig. 52). The dimensions of this hall are 219 ft. for the major axis of the ellipse, and 185 ft. for the minor axis, the circumference of the wall being nearly 700 ft. In order to carry the feet of the trusses and distribute the load equally along the wall, a continuous wrought-iron wall-plate of girder shape, resting with its web on the wall, was adopted. The roof itself is formed of thirty half principals on the trussed girder or bowstring system, of which the lower member, instead of being a simple tie-rod of circular section, is made of an inverted ⊥ form, to enable it to receive compression as well as take tension. By uniting also both the upper and lower members of these principals in central curbs, which are assisted in keeping their form by tie-rods, the principals are enabled to act either as arches or as trussed girders, and thus can better cope with unusual or irregular strains. In considering the stability of such an arrangement, it is to be observed that when the principals act as arches, notwithstanding the inequality of their thrusts, the strain on the wall-plate will not vary correspondingly, owing to the smaller radius of curvature at the end of the longer axis, and the different angles which the lines of tension on the wall-plate therefore make with those of the thrusts. In fact, the calculated thrusts of the principals do not vary materially from those which are necessary to keep the wall-plate in equilibrium. Supposing, however, any unexpected disturbance to arise, the tendency to loss of shape would at once be arrested, for the lower members of the principals would come to the rescue, and get into full work as tie-rods before damage to the walls could occur. There is another element of strength in a roof of this construction which should not be lost si

Each principal is equal to carrying the weights to which it can apparently be subjected, whether acting as an arch or as a trussed girder; but if any weakness existed in a principal it would, from the polygonal form of the purlins, receive much support before mischief was done. The purlins towards the crown must suffer compression, and those on the haunches get into tension as the crown of the roof sinks, and any other change of form would be resisted in a similar manner. Again, the especial defect of girders of this form as respects lateral and sudden strains from wind is here in a great measure obviated; for not only is the pressure upon any particular half-principal transmitted through the purlins to its neighbours, but the movement of the central curb on which such half-principals rest will be resisted by the series of thrusts over a large arc of the wall-plate on the side opposite to that in which the impetus is given. (See General Scott's description of the Albert Hall, in the "Transactions of the Royal Institute of British Architects," for January 22, 1872.) The rise of the principals at the centre of this roof is 50 ft. above the springing, and the depth of each truss at the middle is 18 ft. The boom of the rib is formed with a plate-web, $\frac{5}{16}$ in. thick and 9 ins. deep, with top flange of angle irons $3'' \times 3''$ riveted to the web, a top plate 11 ins. wide being riveted to the angle irons. The straining-pieces are formed of four angle irons $2\frac{1}{2}'' \times 2\frac{1}{2}''$, kept apart by cast-iron distance pieces, which make them swell out in the middle. The cross-bracing between the straining-pieces is formed of rods $1\frac{1}{2}$ in. diameter, riveted to the web of the boom and tie-beam. The tie-beam is formed of a plate-web 9 ins. deep, to the bottom of which two angle irons $3'' \times 3''$ are riveted.

The method of applying the diagrams of strains to

the calculation of the strength of bowstring trusses is shown in Humber's "Handy-book for the Calculation of Strains in Girders." *

44. Another form of iron roof which we have to consider is that in which the principal or boom consists of an arched rib or curved truss. The bowstring truss may be considered as an arched roof, only in that form the depth of the boom is very small as compared with the span of the roof. In the usual form of arched roofs the curved rib is made of sufficient strength to resist the strains to which it is subjected, without the necessity of additional bracing. Owing, however, to the curvature which is given to these ribs, there must of necessity be a certain amount of horizontal thrust upon the supports, which has to be counteracted either by a horizontal tie or by strengthening the abutments.

The roof over the Pimlico Station of the Chatham

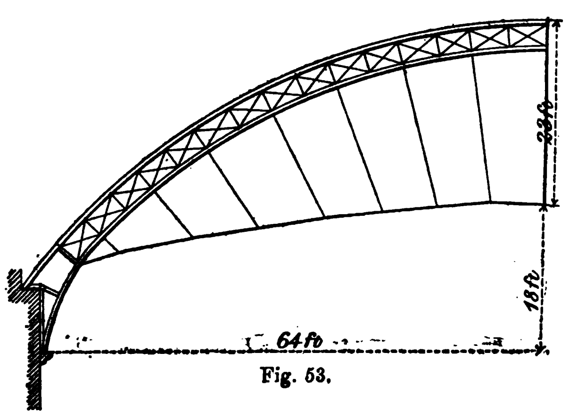

Fig. 53.

and Dover Railway is an example of a curved rib of which the thrust is counteracted by means of a hori-

* Crosby Lockwood & Co.

zontal tie-rod (Fig. 53). The main rib is in the form of a segmental arch, having a span of 129 ft. and struck with a radius of 78 ft. The crown of the arch is 41 ft. above the springing, and the depth of each rib is 4 ft. The ribs are placed 35 ft. apart, and are formed of lattice work, being divided into thirty-two compartments by radiating straining-bars, between each pair of which is a pair of cross braces. The bays at the feet which rest on the columns are curved to a radius of 17 ft., and are filled in with boiler plate, the upper part of which is single and $\frac{1}{2}$ in. thick, while the lower part, which forms the shoe that fits on the cast-iron saddle, is double and of $\frac{3}{8}$ in. plate. The feet which rest on the walls are filled in with single $\frac{1}{2}$ in. plate. The top and bottom booms are of double angle iron, each of which is $5'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$, fastened together by $\frac{3}{4}$ in. rivets. The straining-bars are of double T-iron, each $4\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{2}''$, placed back to back transversely to the rib. The lattice-bars are of $\frac{1}{4}$ in. bar iron, 3 in. wide, and are passed between the double T-iron of the straining-bars and riveted between to the webs of the booms. The thrust of this arch is resisted by tie-rods or chains, consisting of three parallel flat bars, each $4'' \times \frac{1}{2}''$, held up to the rib by radial-rods, so as to form a catenary curve or polygon, having its joints at the junctions of the radial-rods. These tie-rods are connected to the feet of the main rib by two flat bars, each $7'' \times \frac{9}{16}''$, riveted to the boiler-plate above mentioned. The radial-bars are $1\frac{1}{8}$ in. round wrought iron, and are connected by shackles to the lower boom or flange of the main rib. The foot of each main rib which rests on the outer wall is supported by rollers fixed in a frame, by means of which all contraction and expansion arising from changes of temperature are provided against. The purlins between the ribs are

latticed in the same manner as the rib, and have the same depth.

For details of the roof above described the reader is referred to Humber's "Record of Modern Engineering" for 1863.*

We may here observe that lattice work is introduced in preference to the full web-plate rather on account of its lighter and more pleasing appearance than for any actual superiority in strength. In fact, if we compare weight with weight the full web or plate girder is stronger than the lattice beam.

45. The roof over the St. Pancras Station of the Midland Railway, in London, designed by Mr. W. H. Barlow, is also an example of the arched rib having its outward thrust on the supports counterbalanced by means of a horizontal tie. The tie in this case is, however, invisible, as it consists of a series of girders which carry the floor of the station. The span of this roof is 240 ft. in the clear, and each principal is formed of a curved rib in the form of a Tudor or four-centre arch, pointed at the vertex, rising to a height of 102 ft. above the floor; the principals are placed at distances of 29 ft. apart, with three intermediate ribs, and purlins every 18 ft. The rib is of open lattice work, and has a depth of 6 ft., being made of channel iron and plate iron, forming an open box girder 16 ins. wide, each of the booms having a depth of 10 ins. The purlins are 4 ft. 3 ins. deep, of T-section, and braced; they are secured at top and bottom to the upper and lower booms of the main ribs, to which they act as stiffeners. The lower part of the rib is formed with a full plate-web of $\frac{3}{8}$ in. metal, and is widened out to 3 ft. 6 in. at the base, which rests upon a wrought-iron shoe 13 ft. long and 2 ft. 7 ins. deep, bolted down to

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the foundation of the wall. To this shoe is attached the continuous wrought-iron girder which carries the floor and acts as a tie to the arched rib; this girder is of I-section, having a depth of 21 ins. and carried upon fifteen cast-iron columns. In a roof of this arrangement there is no necessity to provide for expansion and contraction by changes of temperature, as the tie being covered up by the ballast would vary to an inconsiderable amount, while the arched rib would be at liberty to rise or fall at the crown without detriment to the structure. The results arrived at in considering the design, are thus enunciated by Mr. Barlow, being obtained partly by calculation and partly by experiment:—

1st. That the depth of the rib must be sufficient to contain all the lines of pressure generated by the dead load, by snow, and by the pressure of the wind.

2nd. That the sectional area of the metal should be sufficient to sustain the whole stress without producing a strain on the iron exceeding 3½ tons per square inch.

3rd. That the arch should be riveted together with proper joint plates throughout, so as to give it the advantage of complete continuity.

The arch is made slightly pointed at the top, partly to give greater resistance to the lateral action of the wind, and partly to produce a better architectural effect, by giving a definite apex to the interior of the roof.

46. When the arched rib is employed for the principal of a roof it is generally adopted in order to avoid the necessity of having any tie-rod across from one foot to the other, and to have the opening clear to the vertex. In this case the thrust produced by the load upon the arch has to be counteracted by strong abut-

ments. As an example of this kind of roof, may be mentioned that over the Metropolitan Railway Station at Kensington, of which the details are given in Humber's "Record of Modern Engineering" for the year 1866.* Each of the principals of this roof consists of an elliptical arch of wrought iron 16 ins. deep at the crown and increasing slightly in depth towards the springing. The arch is a full web girder of I-section, formed of a plate at top and bottom riveted with angle iron to the plate-web. It is secured at the springing to a cast-iron shoe, which is firmly bolted down to a mass of brickwork. The span is 87 ft. and the rise at the centre 29 ft. above the springing. The principals are placed 22 ft. apart.

47. The roof over the Cremorne Music Hall, de-

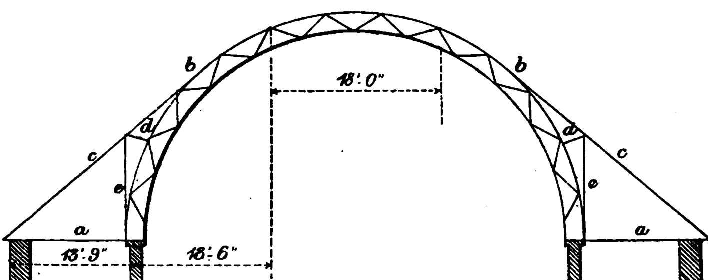

Fig. 54.

signed by W. Humber, C.E., is another example of an arched roof without a tie, and is elevated on pillars a considerable height above the ground. Each principal consists of a semicircular rib 2 ft. deep and having a span of 45 feet; the abutment being provided in the roofs and floors of the galleries on each side, which are 12 ft. wide (Fig. 54).

The arch is formed with a top and bottom boom of

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T-iron, having a width of 4 ins., a depth of $3\frac{1}{2}$ ins., and $\frac{3}{8}$ in. thick. These booms are braced together by diagonals $2\frac{1}{2}'' \times \frac{3}{8}''$; and to insure the stiffness of the lower flange a $\frac{1}{4}''$ plate 8″ wide is riveted to the underside and carried upwards to a height of 18 ft. from the springing. On either side the springing plate *a* of the arch is elongated to the outer walls, upon which it rests and forms the tie-beam of the roof principals over the galleries. To the end of this tie-beam the tension rod *c* is attached and forms a tangent to the arch at *b*, where it is riveted to the outer member, leaving a crown piece of 18 ft. bearing between the supported segments. This external supporting tie is of T-iron, having the same section as the top and bottom booms of the arch. In order to meet any tendency to buckling between the point *b* and the springing, a vertical tie *e* is inserted, which is $2\frac{1}{2}'' \times \frac{3}{8}''$, and from its point of junction with the main tie a short strut of bar iron 4 ins. wide and 1 in. thick is made to rest upon the arch at *d*, thereby forming a complete truss. Thus a very strong roof has been constructed with considerable economy of material. The principals are placed 14 ft. 6 ins. apart, and support twelve purlins of rolled I-section $6\frac{1}{4}'' \times 3''$.

With regard to the strains to which these principals may be subjected, the side portions from the springing to the point *b* may be considered as a cantilever balanced by the weight of the galleries and outer walls; and the intervening 18 ft. of the crown may be regarded as a simple slightly curved triangular girder. The details of the construction of this roof are given in Humber's "Record of Modern Engineering" for the year 1863.*

48. The main roof of the Dublin Winter Palace is

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formed of arched ribs, 17 ft. apart, of lattice work, having a span of 50 feet, the thrust of which is sustained by buttresses (Fig. 55). The ribs are semicircular, having a depth of 18 ins. at the crown and deepened out to 3 ft. at one-third of the height from the springing, at which point commence cast-iron buttresses carried down to the arched roofs over the galleries. These arched girders are cast in two pieces

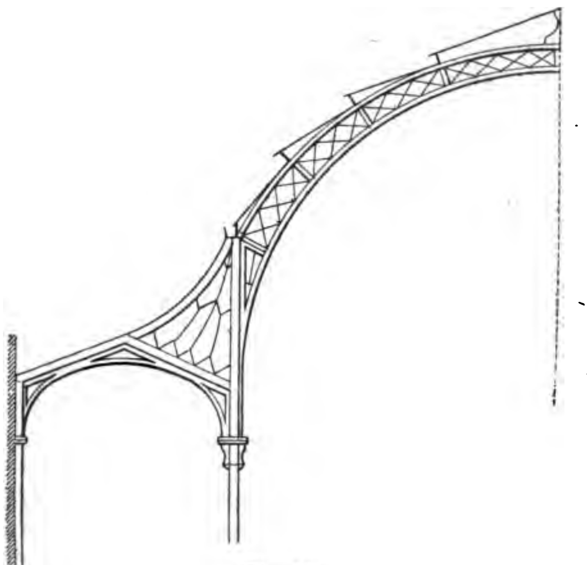

Fig. 55.

firmly connected at the centre and attached to the columns with bolts at the upper part, and by wrought-iron keys at the springing. Below the gallery floors are open transverse girders cast with strong brackets at each end, the latter being intended to reduce the leverage on the columns, as is the case with the arched form of the roof girders. In order to complete the abutment, the bases of the two corresponding columns

are bolted to a girder which forms the bed-plate, and is laid on rubble piers underneath the columns.

The details of this roof will be found in Humber's "Record of Modern Engineering" for 1864.

49. A roof of similar form to that described above, but of much wider span, was erected in 1862 over the Agricultural Hall at Islington. The centre span is 125 ft., and is surmounted by segmental ribs of wrought-iron trellis work with cast-iron struts, which are placed 24 ft. apart, and carry six lines of purlins, also of trellis work, the same depth as the main ribs which they intersect. The roof is secured by wind-ties placed between the principals. The thrust of the arches is taken by the side galleries, which are 36 ft. wide, the roof being carried on a double row of cast-iron columns, whose stability is increased by the floor girders of the galleries.

50. The main roof over the Market Hall at Derby is also formed in a similar manner to the above, the principals consisting of semi-circular ribs with a span of 86 ft. and a uniform depth of 28 ins., placed 24 ft. apart. The arched rib is formed of a web-plate $\frac{5}{16}$ in. thick, with top and bottom flanges of angle irons $3\frac{1}{2}'' \times 3\frac{1}{2}''$ riveted on each side of the web. There are fifteen purlins attached to the sides of the ribs by means of angle plates riveted on to each; these purlins have a depth of 18 inches, and are formed of open lattice work riveted to the two pairs of angle irons which constitute the top and bottom flanges. They carry the wooden rafters on which is laid the boarding to receive the zinc covering. In order to lighten the main ribs, and also to give them somewhat of an ornamental appearance, a series of quatrefoils are formed on the web by punching round holes therein. The feet of the

ribs rest on the top of cast-iron columns 23 ft. in height, which also support an arched roof of iron 11 ft. 6 in. span over the side aisles; these latter form the abutment to counteract the thrust arising from the main roof, and convey it to the outer walls of the building.

51. The arched iron roof over the transept of the Crystal Palace at Sydenham differs very considerably from any of those previously described. In this case the ribs are semicircular, the span being 104 ft. and the height of their springing above the floor 108 ft. In consequence of the exposed situation of this roof it was necessary to give it an unusual amount of strength, so as to enable it to resist any severe strain arising from high winds. Great stiffness has therefore been given to the ribs by making them 8 ft. in depth, so that they become simply arched girders and produce but little if any horizontal thrust upon the supports. Each rib has a top and bottom flange formed of $\frac{1}{4}$ in. iron plate, 10 ins. wide, riveted to double angle irons $6'' \times 3\frac{1}{2}''$, the web consisting of double lattice work of the form XX of flat bar iron, the ends of which are riveted to the angle irons above mentioned, and also to radiating struts which are placed between each set of diagonals, and are riveted to the flanges. The struts themselves are alternately of wrought and cast iron, the latter receiving the purlins, which are 6 ft. in depth, and of double lattice work riveted to flanges in a similar manner to the main ribs. The principals of this roof are 24 ft. apart and are supported upon a double row of cast-iron columns, one under the foot of the upper boom and another under that of the lower boom of the ribs; and the abutment is formed by five tiers of cast-iron trellis girders placed transversely, which carry the floors and roofs of the galleries, and are themselves strongly bolted to the supporting pillars.

52. In roofs of the form we have just described, there is a considerable waste of material if the arched rib is made of uniform strength throughout, as the strain which it has to sustain is much less at the crown of the arch than at its haunches, generally in the proportion of 1 to 2; so that the strength at the crown should be made about one-half of that at the lower part. This is generally done by reducing the depth of the web, and as the strength varies as the square of the depth, the proportion which the depth of the parts should bear to one another is that of 1 to $\sqrt{2}$, or nearly as 5 to 7. A method of investigating the strains on arched roofs will be found in "The Science of Building," 2nd. edit.,* where it is shown that in an arched rib of 40 feet span, loaded in the ordinary manner, the strength of the haunches should be equal to that of a girder of I-section having the depth 12 ins., breadth 8½ ins., and thickness of metal ¾ in. The depth of the crown may be $\frac{5}{7}$ths of this, or 8½ ins., if the breadth and thickness remain the same. Since the load on the roof increases directly as the span, and the leverage increases in like proportion, it follows that the moment of resistance of the beam about its neutral axis must increase as the square of the span, so that in a roof with a span of 80 ft. the moment of resistance must be four times as great as for one of 40 ft. span. As, however, the strength is as the square of the depth, we have only to double the depth if all the other dimensions remain the same as before. In calculating these strains the ribs were supposed to be 10 ft. apart and the total load on the purlins about 42 lbs. per square foot of roofing; with a greater distance apart of the principals their strength must be proportionately increased.

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53. The most remarkable iron roof of recent times is that which was erected by Mr. J. Scott Russell over the Vienna Exhibition building in the year 1873. This roof was conical in form, having a slope of 30° with the horizon, and a span of 360 ft., the base of the cone being elevated 80 ft. from the ground upon 30 columns placed 36 ft. apart. It is formed entirely of 360 iron plates tapering uniformly upwards from the circumference to the apex of the cone and riveted together. The upper portion is cut away, leaving an opening 100 ft. in diameter, over which is erected another conical dome standing on a drum 34 ft. high, and on the top of this a third dome 24 ft. diameter on a drum 28 ft. high, making a total elevation of 280 ft. In such a cone there is great lateral and tensile strength, which entirely prevents any outward thrust from being thrown upon the supports, and its resistance to crushing is so great that a considerable saving in weight or pressure upon the foundations can be effected, the thickness of the plates being diminished from the base upwards, as the compression is greatest at the base and decreases regularly towards the top. The tensile strength at any part is also in proportion to the diameter and therefore to the quantity of metal, being greatest at the base where there is most resistance, and diminishing upwards with the decrease in the quantity of metal. Thus there is no loss of power or material, the maximum of strength being obtained with the minimum of material. Openings to an almost unlimited extent can with safety be made in the cone, so as to form a mere skeleton dome, provided that all the parts remain connected together, and the stability of the structure will remain unaffected. A full description of the construction of this roof is given in a paper read by Mr. Scott Russell before the Royal Institute of British Archite-

on February 9th, 1874, which will be found in the transactions of that society. As the result of investigations made during a period of thirty years, Mr. Russell has come to the conclusion that a certain conic form develops the strength of wrought iron on a large scale of structure so as to attain the maximum possible of strength, economy, and endurance. It is requisite, however, that the tensile and compressive strength of the iron should be equal, and that it should possess great toughness. Good iron has a strength of 20 tons per square inch both for tension and compression, and may at the same time have a high degree of toughness. To demonstrate the advantages of the conical roof over any other form, Mr. Russell first examines the strains borne by the parts of a roof made in any of the ordinary ways, such as we have been describing in the previous pages, and then proceeds to show how the strains act upon the iron cone.

The Common Ways of applying the Material.—Take, first, the best modern way known to us: the iron beam, or straight girder, top and bottom alike, called a rail, a parallel beam, or a bridge girder. Take a pair of high walls and lay these girders across from wall to wall and we get a simplest possible roof, perhaps the strongest of its kind. Now, what is the case of this roof as regards strength? The top is all in compression, the bottom is all in tension; one is in danger of being crushed, the other is in danger of being torn. When either gives way the other's strength is useless; the yielding of the one half leaves the other half worthless. This beam roof consists, then, in two separate halves, which are independent, even contrary in their action, as neither helps the other. But there is also a third—the middle web—which may, in a deep beam, hold as much matter as any of the others. This beam

is also rendered useless the moment the top or bottom has given way. Three things then do different work, in which neither can help the other with spare strength, to supply and make good its weakness; one broken, all three are gone. What, then, is the case of such a beam when breaking?—that only half its strength has been useful. But the case is worse than this, the beam is at least twelve times its depth in length. Now what does this leverage do? Does it help the beam to carry weight? No; it helps the weight to break the beam.

Let us examine the action of weight on our beam. Let us take not half but the whole depth as effectual leverage for strength. The length of lever then is 6 to 1; in other words, the weight on the beam brings a strain on the top tearing the iron asunder with sixfold its natural force per square inch of section. The strain brought on the top is sixfold that amount; also crushing instead of tearing it, neither helps the other nor participates in its strain. What the third part or central web does, is really to help the weight to tear the bottom asunder, and to help the weight to crush the top at the same time. The middle is a mere instrument for setting top and bottom against each other; it is really the fulcrum which ruins the mechanism. Around that centre acts the leverage. The resistance of the top tears the bottom; the resistance of the bottom crushes the top; the middle helps to ruin both.

A lever, then, consists of two parts set against each other in antagonistic attitude to destroy each other; while neither yields all is stable. When the weight, the leverage, and the antagonism united, force one to give in, all give way. This waste of useful force in useless antagonism is characteristic of the whole group of trussed roofs. We take two walls, and instead of laying from one to another the straight girder above

mentioned, we lay across two girders which we may call rafters. We lay them sloping opposite ways in order that by opposing pressures they may balance each other. They do balance, and if the angle is high, the harm is small. As we moderate the height or pitch of the roof, we increase rapidly the waste strain. We soon make it manifold the original weight, and the two beams meeting, push against each other with high destructive power. They also push against the walls they should unite and support, which must be kept from falling by buttresses or more waste. To spare this waste we take a pair of tie-beams, joined in the middle, to undo the harm which the rafters would otherwise do to the walls. A roof truss is only another form of waste, aggravated by the smallness of the slope we select, and doubled by the precautions we invent to undo it.

1*st Principle.* — *Unity without Antagonism.* — This double waste is avoided in the conical roof, where every atom of iron does its own work only. In short, it is a congregation of atoms working together for the common support, without counteraction, disagreement, or waste of strength, or means, or work. This principle of perfect unity of co-operation in the atoms of iron, without antagonism and waste, is one principle and the first peculiarity of the cone. There is a second peculiarity, which grows in importance with the largeness of the scale of the structure. When the scale of a structure is small, its own weight may be the smallest element in its work; when the scale is large, its overweight becomes very rapidly its own heaviest load, and may easily become its own heaviest cause of destruction.

2nd Principle.—Wise distribution of Structural Load. —This principle finds its happiest illustration in the

cone. The distribution of the load of its own structure is best possible. The cone gives an absolute maximum of sustaining power ; when on the large scale this principle rules the structure. As an example of the contrary of this principle, take the elliptic or parabolic form, so frequently, and within certain limits, rightly applied. In this form the depth is proportioned at each point of length to the strain at that point ; this strain is greatest at the centre of length, and least at the ends. This is right on a small scale ; on a large one it commits a grave error, that exactly in the place where a load produces the greatest amount of destructive effect, there is the place chosen to set the greatest quantity of destructive load, and just at each end where weight would produce the smallest amount of injury, that is chosen as the point where least destructive load is laid.

The distribution of the load in the cone is just the contrary of this: its material is lightest at the centre, and heaviest at the outside. We are enabled to remove all the heavy loads off from the points farther from support, and to place all the heaviest weights on the points nearest the vertical support. Thus the strain on the matter is a minimum, and the weight of the material is a minimum, and that minimum is in the place where more would be harmful, and the maximum is in the place where maximum weight does minimum harm. The second principle deserves the greatest attention, because the benefit it confers is twofold, and because it is contrary to that most frequently taught, sanctioned by science, and generally acted on.

3rd Principle.—Co-operation of Independent Parts.—There is a third principle, allied with the other two, in the cone, and perhaps it is the most important of the three, that in every large structure every part of

the whole should be ready and able to render assistance to every other part in the performance of its duty, or in the case of imminent danger. Each part should not merely do its own duty without hindrance from another part, but it should be placed in circumstances which enable it to lend all the reserves of its own strength to each other part that is exposed to severe, excessive, or exceptional trial.

Mechanical Nature of the Iron Cone.—The Crushing Strain.—An iron cone may be examined and tested in two ways: it may be exposed to pressure from within on the hollow side, or to pressure from without on the round side. To understand the conic roof, imagine a ring or cylinder of wrought-iron plate, homogeneously riveted together into one upright plate 200 ft. high and 360 yds. round, and say 1 in. thick. Every yard high of this ring, for each inch round, would weigh 10 lbs. It would therefore take 672 ft. of height to bring a weight of 1 ton over each inch of ring. The maximum strain possible on the lowest and most heavily strained part of the ring is 1 ton on the square inch, which is far within the widest practical margin. Take a ring 200 ft. high, strained no less than one-third of a ton per inch, and conceive it resting on a smooth flat level base; then it will be seen that the strain cannot be made greater or less than simply the weight per square inch due to the upright column of metal standing above each inch of base, and diminishing gradually upwards to the summit. Now suppose this cylinder to be cut vertically into couples of triangles, an upright and an inverted triangle alternately, and let all the inverted triangles be removed, leaving all the others standing upright in a ring, points up, taking away half the whole iron. Next incline all these triangles inwards towards each other, and they all then touch each other along

the edge from heel to point. The triangles, say 30 in number, form the cone. They are slightly curved one way, their upright axes remaining straight when inclined, their curvature being diminished exactly in the ratio of their diminishing width. Suppose the edges to be firmly united together, and the cone is complete.

In this state the matter of the cone is nowhere more or less crushed than it was when it had not yet become a cone, or the crushing strain on it is everywhere less than one-third of a ton per square inch. Now let the thickness of plate be diminished uniformly towards the top, then only one-third of the whole material of the cylinder remains in the cone, and the maximum strain is still everywhere less than one-third of a ton to the inch. The crushing strain along the slope of the cone is therefore extraordinarily small.

The Tearing Strain.—A simple mode to think out the tensile strain of the cone is to study its change of shape, as it is conceived to be flattened out. Take the original formation of the cone, thirty triangles fastened together in one, and lay them out flat by pushing down the top and swelling out the bottom. It will be seen that the radius of the cone base is increased. That all the conic triangles are sundered, the spaces left being triangles all with a common summit; the base has stretched 176 ft., or the vacant space forming the base of the triangles of extension is near 6 ft. If the cone had been of india-rubber, each triangle of 36 ft. material would now have stretched to 42 ft. base. Also it is seen that in exact proportion as the web narrows does the quantity of stretch diminish. Everywhere, therefore, along the web, from outside to centre, the stretch is in the same proportion to the length of web stretched, so that the whole material is in uniform

stretch throughout, and the whole material equally co-operates in resisting this strain. Thus there is neither waste, defect, nor antagonism—the whole cone is in uniform tension, from top to bottom, and all round.

We thus see that the whole cone is a series of straight-lined tapering bars, starting from a circle and meeting in a point, all of them in compression endways by their own crushing weight. We see that at the same moment the whole cone is a series of circular hoops, increasing in thickness as in diameter, and carrying a strain as hoops, which strain is always proportioned to the size and strength of each hoop, and increases uniformly from top to bottom, in size, in strain, in thickness, in strength.

The Combined Strains.—The third important view of this subject consists in looking at these two strengths and strains together. It is the same matter which bears simultaneously the two strains. Each bit of iron is a bit of the straight tapered bar, and stands a crush, and is a bit of the round hoop, and stands a tear. Now these two are neither in same direction so as to conspire, nor in opposite directions so as to counteract each other: one is at right angles to the other; each does its double work without interference. Each atom also is crushed one way, torn another way; the same atoms work two ways at once. This cone then gives the greatest possible amount of resisting power out of a given quantity of matter.

Conic Skeletons.—It is one of the peculiarities of the cone that it is a self-contained structure, and that much of it may be taken away, or injured, or displaced, without destroying it. We may pierce it full of holes all over and it remains stiff, stable, strong, self-supporting. Nothing is harder than to destroy the cone.

We may cut away its centre and it will stand all the same, or cut the outer ring through in many places and it will still stand. All modifications of it are, however, weaker than the cone itself. A skeleton cone may be formed by cutting away a large portion of the surface of the cone, leaving the intermediate parts all connected together, no matter what the size or shape of the apertures, and we have still left a perfectly balanced self-contained structure. These skeleton conic structures are light, strong, and convenient for a multitude of purposes, and are to be preferred, as stronger, cheaper, nobler than common trussed roofs.

As the dome of the Vienna Exhibition had to rest upon columns placed at intervals of 36 ft. without any intermediate support, it became necessary to adopt a combination of the continuous cone with the skeleton cone, in order that it might rely only upon itself for its stability, and also carry all the strains of everything round about it. As each detached column would tend to buckle up the cone at the place where they meet, a *stringer* or straight girder was fixed from the head of each column to the summit. And in order to prevent lateral distortion from the same cause, *ring* girders were placed horizontally all round the circumference of the cone in the manner of purlins, intersecting the straight or *ridge* girders before mentioned. The rings and ridges were placed on the outside of the cone with a view to prevent great masses of frozen snow and ice from sliding off the roof and damaging the surrounding lower buildings on which they fell.

54. The same principle of construction which exists in the conical or straight dome may also be applied to the curved or spherical dome, only in this case it is necessary that the iron plates should have a double curvature, and therefore would require to be hammered

out to the required shape. This would involve a great increase in the labour and cost of construction over the straight dome, but the result would be much more pleasing to the eye, although the strength for the same weight of metal would be less than in the cone. The cost, however, of building a spherical dome of plate iron would be much less than if the material used was brick or stone, and the load upon the foundations would also be considerably reduced. Besides which, there is no outward thrust upon the supporting walls in the iron dome as there is in one of brick or stone, unless it is tied round with strong belts of iron. Domes of iron can also be safely made with a far greater span than domes of brick or stone.

Buildings which are elliptical in plan can also be covered with iron plate domes in the form of an ellipsoid; but the farther a domical roof departs from the sphere in shape the weaker it becomes in proportion.

55. The usual mode of constructing domical roofs over circular or elliptical buildings is either by employing trusses of the bowstring form, as in the case of the roof of the Albert Hall, previously described (43), or else by erecting a number of iron ribs such as those of the St. Pancras Station (45), placed at equal distances all round the building, and connected together by trussed purlins which serve the purpose of tying in the ribs and preventing any outward thrust upon the supports. An example of this kind of domical roof is seen in that over the Reading-room of the British Museum, which has a diameter of 140 ft. and an altitude of 106 ft. The iron ribs which form the skeleton of the dome are twenty in number, and spring from iron stanchions, the space between which is filled in with brickwork.

56. We now proceed to summarize what has been

said in the previous pages with respect to the various forms in which iron can be applied to the construction of roofs.

First, we have the simple *flat roof* supported upon joists or girders of iron, and forming in fact a floor upon which we can walk.

Secondly, we have the *trussed roof*, in which each principal consists of a pair of straight rafters inclined at an angle to the horizon, tied together at their feet with a straight or arched tie-beam, and abutting against each other at the vertex or centre of the roof; the rafters and tie-beam being stiffened and prevented from sagging by means of cross ties, struts, and braces, as in the roof of the House of Lords (35), and that over the Pimlico Station of the Brighton Railway (37).

Thirdly, there is the *bowstring roof*, which is only another form of the *trussed* roof, in which each principal is formed of a single main rafter or boom of a curved or polygonal form, held in at the feet as in the former cases by a tie-beam, with which it is also connected and strengthened by means of struts, braces, and ties. Among this class of roof we have those of the Cannon Street (38) and Charing Cross (39) Stations in London; the New Street Station at Birmingham (41); the station of the North-Western Railway at Liverpool (42); the railway station at Amsterdam (40); and the domical roof of the Albert Hall at Kensington (43).

Fourthly, roofs are formed with *arched ribs* of sufficient depth to take the strains without trussing; the thrust of these ribs being counteracted by horizontal ties; as in the case of the roof of the Chatham and Dover Station at Pimlico (44), where the tie-beam has a curved form, and is held up to the rib by means of suspension rods to prevent it from sagging. The great station roof of the Midland Railway at St. Panc-

London (45), is an example of this kind of roof, the tie-beam being in the form of a horizontal continuous girder laid under the floor of the platform.

Fifthly, *arched roofs* are formed without any horizontal ties, the thrust being all taken by the abutments; of which class is the roof of the station of the Metropolitan Railway at Kensington (46); that over the hall at Cremorne, Chelsea (47); that of the Dublin Winter Palace (48); that of the Agricultural Hall at Islington (49); that over the central transept of the Crystal Palace at Sydenham (51); and that of the Market Hall at Derby (50).

Sixthly, we have *domical iron roofs* either formed of arched ribs without a tie, as in the Reading-room of the British Museum (55); or of trussed ribs, as in the roof of the Albert Hall (43); or the conical roof of iron plates, as in the dome of the Exhibition building at Vienna (53).

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